Brown Buntin Associates, Inc
FHWA-RD-77-108
Calculation Sheets
October 19, 2006

Project #: 05-066
Description: Future WP SP
Ldn/Cnel: Ldn
Site Type: Soft

			Distance to Traffic Noise Contours				
Segment	**Roadway Name**	**Segment Description**	**55.0**	**60.0**	**65.0**	**70.0**	**0.0**
1	Baseline Road	East of County Line	1177	546	254	118	0
2	Baseline Road	East of Locust Road	1178	547	254	118	0
3	Baseline Road	East of Brewer Road	1185	550	255	119	0
4	Baseline Road	East of Palladay	1266	587	273	127	0
5	Baseline Road	East of 16th Street	1254	582	270	125	0
6	Baseline Road	East of 12th Avenue	1361	632	293	136	0
7	Baseline Road	East of Watt Avenue	1215	564	262	121	0
8	Baseline Road	East of Dyer Lane	1266	587	273	127	0
9	Walerga Road	South of Baseline Roac	916	425	197	92	0
10	Watt Avenue	South of Baseline Roac	969	450	209	97	0
11	Watt Avenue	South of Dyer Lane	1082	502	233	108	0
0	0	0	0	0	0	0	0.0

Brown Buntin Associates, Inc
FHWA-RD-77-108
Calculation Sheets
October 25, 2006

Project #: 05-066
Description: Fut NP Offsite
Ldn/Cnel: Ldn
Site Type: Soft

Contour Levels (dB) 55 60 65 70

Segment	Roadway Name	Segment Description	ADT	Day %	Eve %	Night %	Truck % Med	Truck % Hvy	Speed mph	Dist ft	Offset dB
1	Baseline Road	East of County Line	39,300	90		10	3	2	55	75	0
2	Walerga Road	South of Baseline Road	43,600	90		10	3	2	50	75	
3	Walerga Road	North of PFE Road	43,300	90		10	3	2	50	75	
4	PFE Road	East of Walerga Road	17,900	90		10	3	2	45	75	
5	Watt Avenue	North of Elverta Road	56,500	90		10	3	2	45	75	
6	Watt Avenue	North of Antelope Road	42,400	90		10	3	2	45	75	-5
7	Watt Avenue	North of Elkhorn Road	65,700	90		10	3	2	45	75	-5
8	Walerga Road	North of Elverta Road	45,000	90		10	3	2	45	75	-5
9	Walerga Road	North of Antelope Road	44,800	90		10	3	2	45	75	-5
10	Walerga Road	North of Elkhorn Road	34,400	90		10	3	2	45	75	
11	16th Street	North of Elverta Road	9,000	90		10	3	2	40	75	
12	Watt Avenue	North of I-80	86,200	90		10	3	2	35	75	
13	Sorento Road	North of Elverta Road	18,500	90		10	3	2	35	75	
14	Elwyn Road	North of Elverta Road	16,800	90		10	3	2	55	75	
15	Locust Road	South of Baseline	13,600	90		10	3	2	35	75	
16	Pleasant Grove	North of County Line	18,200	90		10	3	2	35	75	
17	Locust Road	North of County Line	12,500	90		10	3	2	35	75	
18	16th Street	South of Elverta	7,000	90		10	3	2	35	75	
19	Dry Creek	North of Elkhorn	18,100	90		10	3	2	35	75	
20	Dry Creek	South of Elkhorn	21,000	90		10	3	2	35	75	
21	Elkhorn	Watt to Walerga	46,800	90		10	3	2	45	75	-5
22	Elkhorn	Walerga to Roseville	78,400	90		10	3	2	45	75	-5
23	Blue Oaks Blvd	Hayden to Fiddyment	17400	90		10	3	2	45	75	
0		Fiddyment to Woodcreek	35500	90		10	3	2	45	75	
0		Woodcreek to Foothills	60200	90		10	3	2	45	75	
0		Foothills to Industrial	55400	90		10	3	2	45	75	
1	Pleasant Grove Blvd	Hayden to Fiddyment	21200	90		10	3	2	45	75	
0		Woodcreek to Foothills	40600	90		10	3	2	45	75	
0		Foothills to Industrial	49200	90		10	3	2	45	75	
0		East of Industrial	44000	90		10	3	2	45	75	
1	Junction Blvd	Baseline to Woodcreek	17300	90		10	3	2	45	75	
0		Woodcreek to Country Club	15900	90		10	3	2	45	75	
0		County Club to Foothills	15800	90		10	3	2	45	75	
1	Baseline Road	Fiddyment to Woodcreek	38500	90		10	3	2	45	75	
0		Woodcreek to Foothills	29600	90		10	3	2	45	75	
0		Foothills to Industrial	11400	90		10	3	2	45	75	
1	Fiddyment Road	Baseline to Village Green	24600	90		10	3	2	45	75	
0		Village Green to Blueoak	26000	90		10	3	2	45	75	
0		Blueoak to Hayden	17600	90		10	3	2	45	75	
1	Woodcreek Oaks	Baseline to Pleasant Grove	15600	90		10	3	2	45	75	
0		Pleasant Grove to Blueoaks	23500	90		10	3	2	45	75	
1	Foothills Blvd	Vineyard to Baseline	51100	90		10	3	2	45	75	
0		Baseline to Junction	46900	90		10	3	2	45	75	
0		North of Blueoaks	29300	90		10	3	2	45	75	
0						0					

Brown Buntin Associates, Inc
FHWA-RD-77-108
Calculation Sheets
October 25, 2006

Project #: 05-066
Description: Future + Project Offsite
Ldn/Cnel: Ldn
Site Type: Soft

Contour Levels (dB) 55 60 65 70

Segment	Roadway Name	Segment Description	ADT	Day %	Eve %	Night %	Truck % Med	Truck % Hvy	Speed mph	Dist ft	Offset dB
1	Baseline Road	East of County Line	45,000	90		10	3	2	55	75	0
2	Walerga Road	South of Baseline Road	38,600	90		10	3	2	50	75	
3	Walerga Road	North of PFE Road	43,000	90		10	3	2	50	75	
4	PFE Road	East of Walerga Road	16,300	90		10	3	2	45	75	
5	Watt Avenue	North of Elverta Road	63,100	90		10	3	2	45	75	
6	Watt Avenue	North of Antelope Road	45,800	90		10	3	2	45	75	-5
7	Watt Avenue	North of Elkhorn Road	68,800	90		10	3	2	45	75	-5
8	Walerga Road	North of Elverta Road	46,900	90		10	3	2	45	75	-5
9	Walerga Road	North of Antelope Road	46,200	90		10	3	2	45	75	-5
10	Walerga Road	North of Elkhorn Road	35,300	90		10	3	2	45	75	
11	16th Street	North of Elverta Road	22,300	90		10	3	2	40	75	
12	Watt Avenue	North of I-80	86,100	90		10	3	2	35	75	
13	Sorento Road	North of Elverta Road	19,500	90		10	3	2	35	75	
14	Elwyn Road	North of Elverta Road	19,900	90		10	3	2	55	75	
15	Locust Road	South of Baseline	5,500	90		10	3	2	35	75	
16	Pleasant Grove	North of County Line	19,100	90		10	3	2	35	75	
17	Locust Road	North of County Line	17,100	90		10	3	2	35	75	
18	16th Street	South of Elverta	12,900	90		10	3	2	35	75	
19	Dry Creek	North of Elkhorn	23,300	90		10	3	2	35	75	
20	Dry Creek	South of Elkhorn	25,700	90		10	3	2	35	75	
21	Elkhorn	Watt to Walerga	47,000	90		10	3	2	45	75	-5
22	Elkhorn	Walerga to Roseville	80,400	90		10	3	2	45	75	-5
23	Blue Oaks Blvd	Hayden to Fiddyment	19500	90		10	3	2	45	75	
0		Fiddyment to Woodcreek	35000	90		10	3	2	45	75	
0		Woodcreek to Foothills	62300	90		10	3	2	45	75	
0		Foothills to Industrial	56400	90		10	3	2	45	75	
1	Pleasant Grove Blvd	Hayden to Fiddyment	22500	90		10	3	2	45	75	
0		Woodcreek to Foothills	43600	90		10	3	2	45	75	
0		Foothills to Industrial	50200	90		10	3	2	45	75	
0		East of Industrial	44800	90		10	3	2	45	75	
1	Junction Blvd	Baseline to Woodcreek	26000	90		10	3	2	45	75	
0		Woodcreek to Country Club	22500	90		10	3	2	45	75	
0		County Club to Foothills	21700	90		10	3	2	45	75	
1	Baseline Road	Fiddyment to Woodcreek	56400	90		10	3	2	45	75	
0		Woodcreek to Foothills	31900	90		10	3	2	45	75	
0		Foothills to Industrial	12300	90		10	3	2	45	75	
1	Fiddyment Road	Baseline to Village Green	24900	90		10	3	2	45	75	
0		Village Green to Blueoak	26100	90		10	3	2	45	75	
0		Blueoak to Hayden	17300	90		10	3	2	45	75	
1	Woodcreek Oaks	Baseline to Pleasant Grove	19300	90		10	3	2	45	75	
0		Pleasant Grove to Blueoaks	24600	90		10	3	2	45	75	
1	Foothills Blvd	Vineyard to Baseline	51700	90		10	3	2	45	75	
0		Baseline to Junction	47600	90		10	3	2	45	75	
0		North of Blueoaks	29300	90		10	3	2	45	75	
0						0					

Brown Buntin Associates, Inc
FHWA-RD-77-108
Calculation Sheets
October 25, 2006

Project #: 05-066
Description: Existing + Proj SP Blueprint
Ldn/Cnel: Ldn
Site Type: Soft

Contour Levels (dB) 55 60 65 70

Segment	Roadway Name	Segment Description	ADT	Day %	Eve %	Night %	Truck % Med	Truck % Hvy	Speed mph	Dist ft	Offset dB
1	Baseline Road	East of County Line	16,600	90		10	3	2	55	75	0
2	Baseline Road	East of Locust Road	16,800	90		10	3	2	55	75	
3	Baseline Road	East of Brewer Road	18,800	90		10	3	2	55	75	
4	Baseline Road	East of Palladay	23,600	90		10	3	2	55	75	
5	Baseline Road	East of 16th Street	25,900	90		10	3	2	55	75	
6	Baseline Road	East of 12th Avenue	29,300	90		10	3	2	55	75	
7	Baseline Road	East of Watt Avenue	32,500	90		10	3	2	55	75	
8	Baseline Road	East of Dyer Lane	35,200	90		10	3	2	55	75	
9	Walerga Road	South of Baseline Road	15,900	90		10	3	2	50	75	
10	Watt Avenue	South of Baseline Road	12,500	90		10	3	2	50	75	
11	Watt Avenue	South of Dyer Lane	43,700	90		10	3	2	45	75	
0											

Brown Buntin Associates, Inc
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Calculation Sheets
October 19, 2006

Project #: 05-066
Description: Existing + Proj SP Blueprint
Ldn/Cnel: Ldn
Site Type: Soft

			Distance to Traffic Noise Contours				
Segment	**Roadway Name**	**Segment Description**	**55.0**	**60.0**	**65.0**	**70.0**	**0.0**
1	Baseline Road	East of County Line	605	281	130	61	0
2	Baseline Road	East of Locust Road	610	283	131	61	0
3	Baseline Road	East of Brewer Road	658	305	142	66	0
4	Baseline Road	East of Palladay	765	355	165	77	0
5	Baseline Road	East of 16th Street	814	378	175	81	0
6	Baseline Road	East of 12th Avenue	884	410	190	88	0
7	Baseline Road	East of Watt Avenue	947	440	204	95	0
8	Baseline Road	East of Dyer Lane	999	464	215	100	0
9	Walerga Road	South of Baseline Roac	507	235	109	51	0
10	Watt Avenue	South of Baseline Roac	432	200	93	43	0
11	Watt Avenue	South of Dyer Lane	849	394	183	85	0
0	0	0	0	0	0	0	0.0
0	0	0	0	0	0	0	0.0

Brown Buntin Associates, Inc
FHWA-RD-77-108
Calculation Sheets
October 25, 2006

Project #: 05-066 **Contour Levels (dB)** 55 60 65 70
Description: Exis + Proj Offsite Blue
Ldn/Cnel: Ldn
Site Type: Soft

Segment	Roadway Name	Segment Description	ADT	Day %	Eve %	Night %	Truck % Med	Truck % Hvy	Speed mph	Dist ft	Offset dB
1	Baseline Road	East of County Line	16,600	90		10	3	2	55	75	0
2	Walerga Road	South of Baseline Road	15,900	90		10	3	2	50	75	
3	Walerga Road	North of PFE Road	15,200	90		10	3	2	45	75	
4	PFE Road	East of Walerga Road	10,600	90		10	3	2	45	75	
5	Watt Avenue	North of Elverta Road	50,400	90		10	3	2	45	75	
6	Watt Avenue	North of Antelope Road	49,600	90		10	3	2	45	75	-5
7	Watt Avenue	North of Elkhorn Road	50,500	90		10	3	2	45	75	-5
8	Walerga Road	North of Elverta Road	22,600	90		10	3	2	45	75	-5
9	Walerga Road	North of Antelope Road	42,600	90		10	3	2	45	75	-5
10	Walerga Road	North of Elkhorn Road	33,400	90		10	3	2	45	75	
11	16th Street	North of Elverta Road	14,600	90		10	3	2	40	75	
12	Watt Avenue	North of I-80	62,800	90		10	3	2	45	75	
13	Sorento Road	North of Elverta Road	4500	90		10	3	2	45	75	
14	Elwyn Road	North of Elverta Road	9100	90		10	3	2	45	75	
15	Locust Road	South of Baseline	1300	90		10	3	2	45	75	
16	Pleasant Grove	North of County Line	4300	90		10	3	2	45	75	
17	Locust Road	North of County Line	9100	90		10	3	2	45	75	
18	16th Street	South of Elverta	6000	90		10	3	2	45	75	
19	Dry Creek	North of Elkhorn	10000	90		10	3	2	45	75	
20	Dry Creek	South of Elkhorn	12200	90		10	3	2	45	75	
21	Elkhorn	Watt to Walerga	27700	90		10	3	2	45	75	-5
22	Elkhorn	Walerga to Roseville	53000	90		10	3	2	45	75	-5
23	Blueoaks Blvd	Fiddyment to Woodcreek	9000	90		10	3	2	45	75	
0		Woodcreek to Foothills	29000	90		10	3	2	45	75	
0		Foothills to Industrial	40700	90		10	3	2	45	75	
1	Pleasant Grove Blv	Fiddyment to Woodcreek	7300	90		10	3	2	45	75	
0		Woodcreek to Foothills	23600	90		10	3	2	45	75	
0		Foothills to Industrial	42100	90		10	3	2	45	75	
1	Junction Blvd	Woodcreek to Foothills	7300	90		10	3	2	45	75	
0		Foothills to Industrial	15700	90		10	3	2	45	75	
1	Baseline Road	Fiddyment to Junction	31700	90		10	3	2	45	75	
0		Junction to Woodcreek	24800	90		10	3	2	45	75	
0		Woodcreek to Country Club	26900	90		10	3	2	45	75	
0		Country Club to Foothills	29400	90		10	3	2	45	75	
0		Foothills to Industrial	11800	90		10	3	2	45	75	
1	Fiddyment Road	Baseline to Blue Oaks	25300	90		10	3	2	45	75	
2	Woodcreek Oaks	Baseline to Pleasant Grove	11100	90		10	3	2	45	75	
0		Pleasant Grove to Blue Oak:	10400	90		10	3	2	45	75	
1	Foothills Blvd	Baseline to Pleasant Grove	28200	90		10	3	2	45	75	
0		Pleasant Grove to Blue Oak:	11900	90		10	3	2	45	75	
0		North of Blue Oaks	3400	90		10	3	2	45	75	
0						0					

Brown Buntin Associates, Inc
FHWA-RD-77-108
Calculation Sheets
October 25, 2006

Project #: 05-066
Description: Fut + Proj SP Blue
Ldn/Cnel: Ldn
Site Type: Soft

Contour Levels (dB) 55 60 65 70

Segment	**Roadway Name**	**Segment Description**	**ADT**	**Day %**	**Eve %**	**Night %**	**Truck % Med**	**Truck % Hvy**	**Speed mph**	**Dist ft**	**Offset dB**
1	Baseline Road	East of County Line	48,200	90		10	3	2	55	75	
2	Baseline Road	East of Locust Road	48,700	90		10	3	2	55	75	
3	Baseline Road	East of Brewer Road	48,300	90		10	3	2	55	75	
4	Baseline Road	East of Palladay	52,800	90		10	3	2	55	75	
5	Baseline Road	East of 16th Street	52,800	90		10	3	2	55	75	
6	Baseline Road	East of 12th Avenue	59,500	90		10	3	2	55	75	
7	Baseline Road	East of Watt Avenue	49,700	90		10	3	2	55	75	
8	Baseline Road	East of Dyer Lane	51,800	90		10	3	2	55	75	
9	Walerga Road	South of Baseline Road	39,700	90		10	3	2	50	75	
10	Watt Avenue	South of Baseline Road	42,500	90		10	3	2	50	75	
11	Watt Avenue	South of Dyer Lane	66,400	90		10	3	2	45	75	
0						0					
0						0					

Brown Buntin Associates, Inc
FHWA-RD-77-108
Calculation Sheets

October 19, 2006

Project #: 05-066
Descriptioı Fut + Proj SP Blue
Ldn/Cnel: Ldn
Site Type: Soft

			Distance to Traffic Noise Contours				
Segment	**Roadway Name**	**Segment Description**	**55.0**	**60.0**	**65.0**	**70.0**	**0.0**
1	Baseline Road	East of County Line	1232	572	265	123	0
2	Baseline Road	East of Locust Road	1240	576	267	124	0
3	Baseline Road	East of Brewer Road	1234	573	266	123	0
4	Baseline Road	East of Palladay	1309	608	282	131	0
5	Baseline Road	East of 16th Street	1309	608	282	131	0
6	Baseline Road	East of 12th Avenue	1418	658	305	142	0
7	Baseline Road	East of Watt Avenue	1257	584	271	126	0
8	Baseline Road	East of Dyer Lane	1292	600	278	129	0
9	Walerga Road	South of Baseline Roac	933	433	201	93	0
10	Watt Avenue	South of Baseline Roac	977	453	210	98	0
11	Watt Avenue	South of Dyer Lane	1122	521	242	112	0

Brown Buntin Associates, Inc
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Project #: 05-066
Description: Fut + Proj Offsite Blue
Ldn/Cnel: Ldn
Site Type: Soft

Contour Levels (dB) 55 60 65 70

Segment	Roadway Name	Segment Description	ADT	Day %	Eve %	Night %	Truck % Med	Truck % Hvy	Speed mph	Dist ft	Offset dB
1	Baseline Road	East of County Line	48,200	90		10	3	2	55	75	0
2	Walerga Road	South of Baseline Road	39,700	90		10	3	2	50	75	
3	Walerga Road	North of PFE Road	43,400	90		10	3	2	50	75	
4	PFE Road	East of Walerga Road	16,700	90		10	3	2	45	75	
5	Watt Avenue	North of Elverta Road	64,400	90		10	3	2	45	75	
6	Watt Avenue	North of Antelope Road	46,900	90		10	3	2	45	75	-5
7	Watt Avenue	North of Elkhorn Road	69,900	90		10	3	2	45	75	-5
8	Walerga Road	North of Elverta Road	47,300	90		10	3	2	45	75	-5
9	Walerga Road	North of Antelope Road	46,700	90		10	3	2	45	75	-5
10	Walerga Road	North of Elkhorn Road	35,200	90		10	3	2	45	75	
11	16th Street	North of Elverta Road	23,100	90		10	3	2	40	75	
12	Watt Avenue	North of I-80	87,000	90		10	3	2	35	75	
13	Sorento Road	North of Elverta Road	20,300	90		10	3	2	35	75	
14	Elwyn Road	North of Elverta Road	20,700	90		10	3	2	55	75	
15	Locust Road	South of Baseline	6,600	90		10	3	2	35	75	
16	Pleasant Grove	North of County Line	20,000	90		10	3	2	35	75	
17	Locust Road	North of County Line	18,200	90		10	3	2	35	75	
18	16th Street	South of Elverta	14,000	90		10	3	2	35	75	
19	Dry Creek	North of Elkhorn	24,500	90		10	3	2	35	75	
20	Dry Creek	South of Elkhorn	27,100	90		10	3	2	35	75	
21	Elkhorn	Watt to Walerga	46,800	90		10	3	2	45	75	-5
22	Elkhorn	Walerga to Roseville	81,400	90		10	3	2	45	75	-5
23	Blue Oaks Blvd	Hayden to Fiddyment	21300	90		10	3	2	45	75	
0		Fiddyment to Woodcreek	36900	90		10	3	2	45	75	
0		Woodcreek to Foothills	64400	90		10	3	2	45	75	
0		Foothills to Industrial	57300	90		10	3	2	45	75	
1	Pleasant Grove Blvd	Hayden to Fiddyment	22900	90		10	3	2	45	75	
0		Woodcreek to Foothills	44600	90		10	3	2	45	75	
0		Foothills to Industrial	51400	90		10	3	2	45	75	
0		East of Industrial	45800	90		10	3	2	45	75	
1	Junction Blvd	Baseline to Woodcreek	28200	90		10	3	2	45	75	
0		Woodcreek to Country Club	25700	90		10	3	2	45	75	
0		County Club to Foothills	24700	90		10	3	2	45	75	
1	Baseline Road	Fiddyment to Woodcreek	61300	90		10	3	2	45	75	
0		Woodcreek to Foothills	32700	90		10	3	2	45	75	
0		Foothills to Industrial	12400	90		10	3	2	45	75	
1	Fiddyment Road	Baseline to Village Green	25100	90		10	3	2	45	75	
0		Village Green to Blueoak	26300	90		10	3	2	45	75	
0		Blueoak to Hayden	17600	90		10	3	2	45	75	
1	Woodcreek Oaks	Baseline to Pleasant Grove	20600	90		10	3	2	45	75	
0		Pleasant Grove to Blueoaks	24700	90		10	3	2	45	75	
1	Foothills Blvd	Vineyard to Baseline	51800	90		10	3	2	45	75	
0		Baseline to Junction	47600	90		10	3	2	45	75	
0		North of Blueoaks	29400	90		10	3	2	45	75	

Additions to Appendix R

Technical Memorandum

SOUTH PLACER WASTEWATER AUTHORITY (SPWA) WASTEWATER AND RECYCLED WATER SYSTEMS EVALUATION PROJECT

Subject: **Trunk Sewer Hydraulic Analysis -- Final (TM No. 3b)**

Prepared For: Art O'Brien – City of Roseville

Prepared by: Pete Bellows/Chris Peters – Brown and Caldwell

Reviewed by: Dave Richardson/Gisa Ju – RMC

Date: April 13, 2006

Reference: 0091-004 Task 3

1 Introduction

This technical memorandum (TM) summarizes the results of the hydraulic assessment of the SPWA collection system and the needed hydraulic improvements, using a set of assumptions and criteria for identifying constraints in the system under conservative design conditions. The hydraulic assessment and development of hydraulic improvements is based on the hydraulic model of trunk sewers in the SPWA collection system. The results of the hydraulic assessment are used to develop potential capital improvement projects. The potential projects provide SPWA with a starting point for evaluating the magnitude of regional trunk sewer system improvements, but need to be refined with site specific field and engineering evaluations.

The hydraulic assessment is based on the flow projections for the ultimate service area presented in TM 2b (Dry Weather Flow Projection) and TM 2c (Wet Weather Flow Projection).

2 Hydraulic Model Development

The development of the hydraulic model network included the development of pipeline network information, development of pump station and force main information, selection of input nodes and delineation of input node tributary areas.

2.1 Collection System Model

The hydraulic modeling program used for this project is H_2OMAP Sewer Pro, a product of MWH Soft, Inc. This software is widely used for hydraulic analysis of collection systems. It is also being used by the City of Roseville to model its collection system. Using the same software for this project allows the information developed for the City's model to be directly utilized for this model.

2.2 Model Network Development

The current network of sewer pipes for the hydraulic model includes all trunk sewers in the SPWA member agencies service areas that are greater than or equal to 15 inches in diameter. They are included in the hydraulic model to provide a complete analysis of the regional collection system. The current model network is shown on Figure 1.

Approximately 83 miles of gravity trunk sewers, 3 pump stations and 4 miles of force main are included in the current model network. The future network includes approximately 100 miles of trunk sewer pipe, 10 pump stations and 19 miles of force main. Alignments of future pipes included in the model network were based upon USGS topographic contour data and available data from the West Roseville Specific

Plan, and planning documents supporting development within Urban Growth Areas (UGAs) such as Placer Vineyards, Regional University and Placer Ranch. The future model network is shown on Figure 2. The amount of pipeline included in the model network is summarized by diameter in Table 1 (gravity pipe) and Table 3 (force main pipe).

Table 1 – Model Network Gravity Pipe Summary

	Length (ft)	
Diameter (in)	Current	Future
6	462	462
8	8,026	8,026
10	8,264	8,264
12	11,405	21,726
15	93,622	102,540
16	8,599	8,539
18	93,644	93,573
20	4,080	4,080
21	33,527	62,190
24	32,801	41,097
27	9,608	23,655
30	26,311	33,570
33	21,508	25,242
36	16,631	19,618
42	30,417	33,494
48	403	4,860
63	8,629	8,629
66	11,286	11,286
72	10,867	10,867
78	5,862	5,862
90	1,082	1,082
Total	**437,035**	**528,663**

Information on the existing trunk sewers was obtained from several sources. Information on the trunk sewers in Roseville was obtained from the City's hydraulic model of its collection system. The City's hydraulic model network was based on the City's GIS of the collection system, as-built drawings, survey data, and discussions with City staff. Trunk sewer information for the Granite Bay (SMD-2) area was obtained from Placer County's sewer GIS. Record drawings were reviewed to develop the model network in SPMUD. Manhole numbers used in the model were provided by each respective SPWA member agency. In some instances there were duplicate manhole numbers along the borders between Roseville and Placer County. In this case, Roseville's manhole numbers were used. For reference, duplicate Placer County manhole numbers are shown on the project summary tables in Attachment C.

Survey information was obtained for some missing invert and rim elevation data. The survey information included the rim elevation and invert elevation of each connecting trunk sewer. Survey information was also obtained at several locations to verify elevation information from other sources. In some areas, pipe invert elevations were estimated using features within H_2OMAP Sewer Pro. Based on given slopes, H_2OMAP Sewer Pro can interpolate invert elevations based on elevations upstream and downstream of the pipe reaches that are missing invert data. This is a very useful modeling feature, but does not always yield accurate elevations in the model. This is the type of data that needs to be field verified by SPWA members as the members proceed forward to identify precise capital projects.

The current network was extended to include the proposed gravity trunk sewers and force mains serving the future development areas in Roseville and Placer County on the west side of the 2005 Service Area boundary. UGAs located to the west of the 2005 Service Area will require pump stations and force mains to convey their wastewater to the SPWA treatment plants. The proposed sewers and force mains are based on preliminary sewer alignments provided in specific plans and USGS topographic contour information. Most UGAs and future development areas were connected (loaded) to existing trunk sewers. There are no specific planned developments identified within the SPMUD service area at this time, and future sewer extensions were not included in the SPMUD collection system at SPMUD's request. The model network for the buildout scenario is shown on Figure 2.

The industry standard Manning's friction factor of 0.013 was assigned to each modeled pipe reach. Manning's friction factor is used by H_2OMAP Sewer Pro to determine pipe hydraulic capacities.

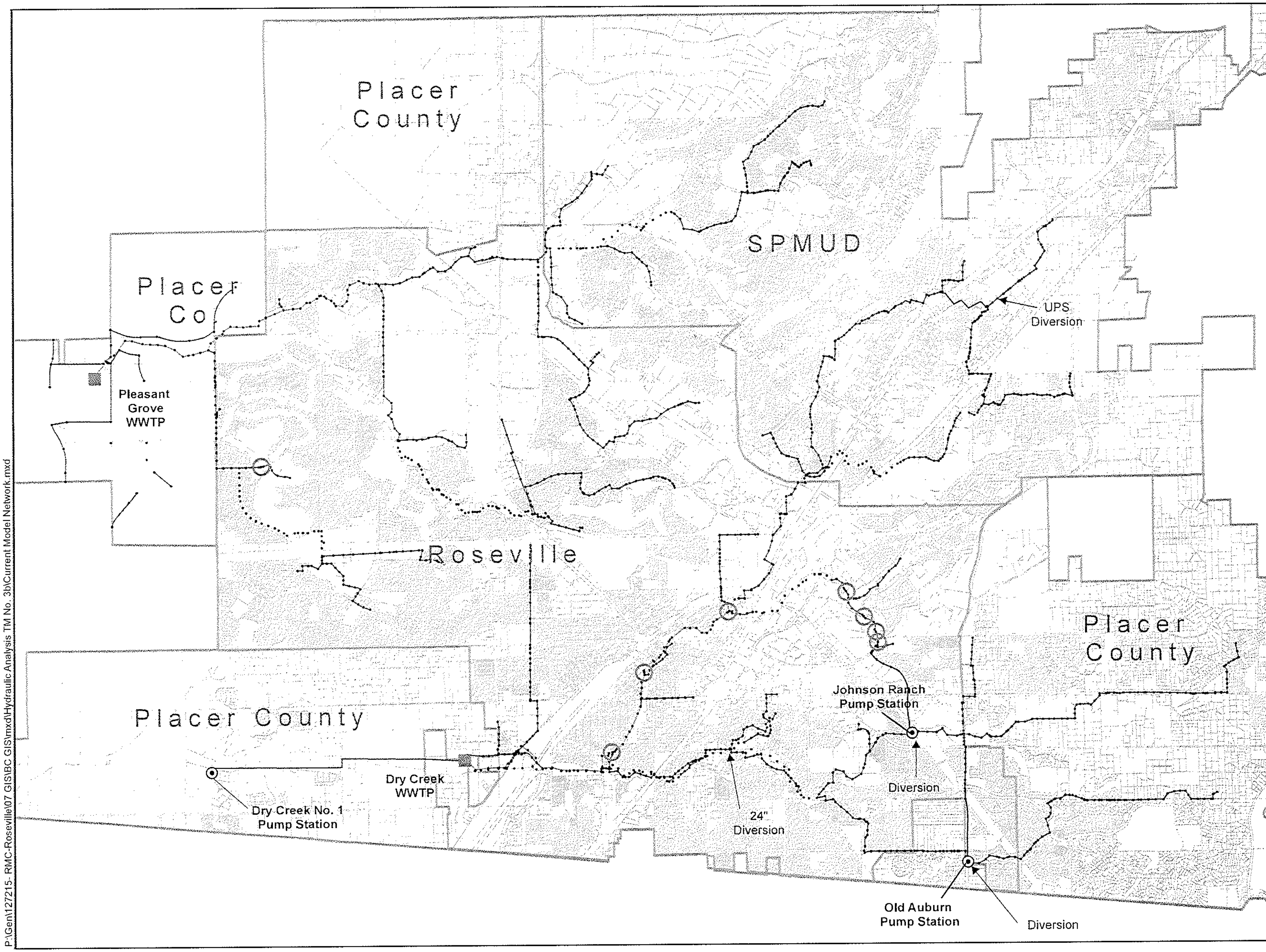

FIGURE 1

Current Model Network

March 2006

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Regional Wastewater and Recycled Water Sewer Evaluation Project

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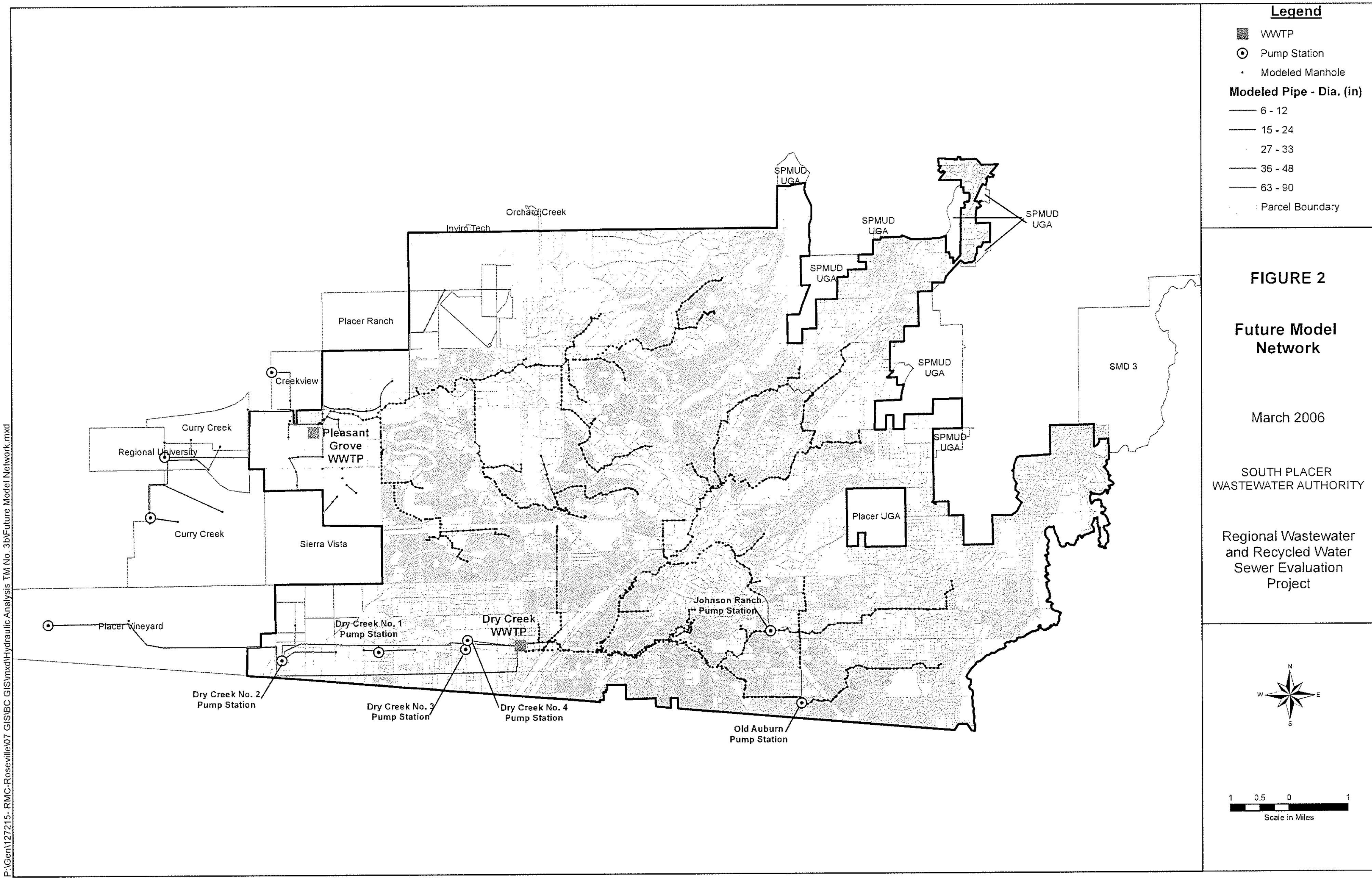

FIGURE 2

Future Model Network

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Regional Wastewater and Recycled Water Sewer Evaluation Project

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2.3 Flow Diversions

The model network includes four flow diversions which are shown on Figure 1. Three of the diversions are located in Roseville and one diversion is located in SPMUD. H_2OMAP Sewer Pro has several options for designating the operation of flow diversions. The UPS diversion in SPMUD has movable boards that direct all flow one direction or the other. In the hydraulic model, this diversion was simulated by putting all flow into the 10-inch diversion pipeline (west) or into the trunk sewer downstream (south) of the diversion. The UPS diversion only affects flow in the SPMUD system. The two diversions upstream of the pump stations in Roseville (Old Auburn and Johnson Ranch) were simulated in the model by allowing an amount of flow equal to the pump station capacity to divert from the trunk sewer to the pump station. The final diversion, at manhole B06-169 in Roseville, diverts flow from the local 15-inch sewer through the 24-inch diversion pipe, to the 33-inch trunk sewer. This diversion is controlled with a weir structure at the manhole. According to City of Roseville field crews that investigated the diversion, approximately 70 percent of the flow is diverted to the 33-inch trunk sewer and 30 percent remains in the local sewer. This diversion was simulated in the model in the same manner. The diversion structure at manhole B06-169 is illustrated in Figure 3.

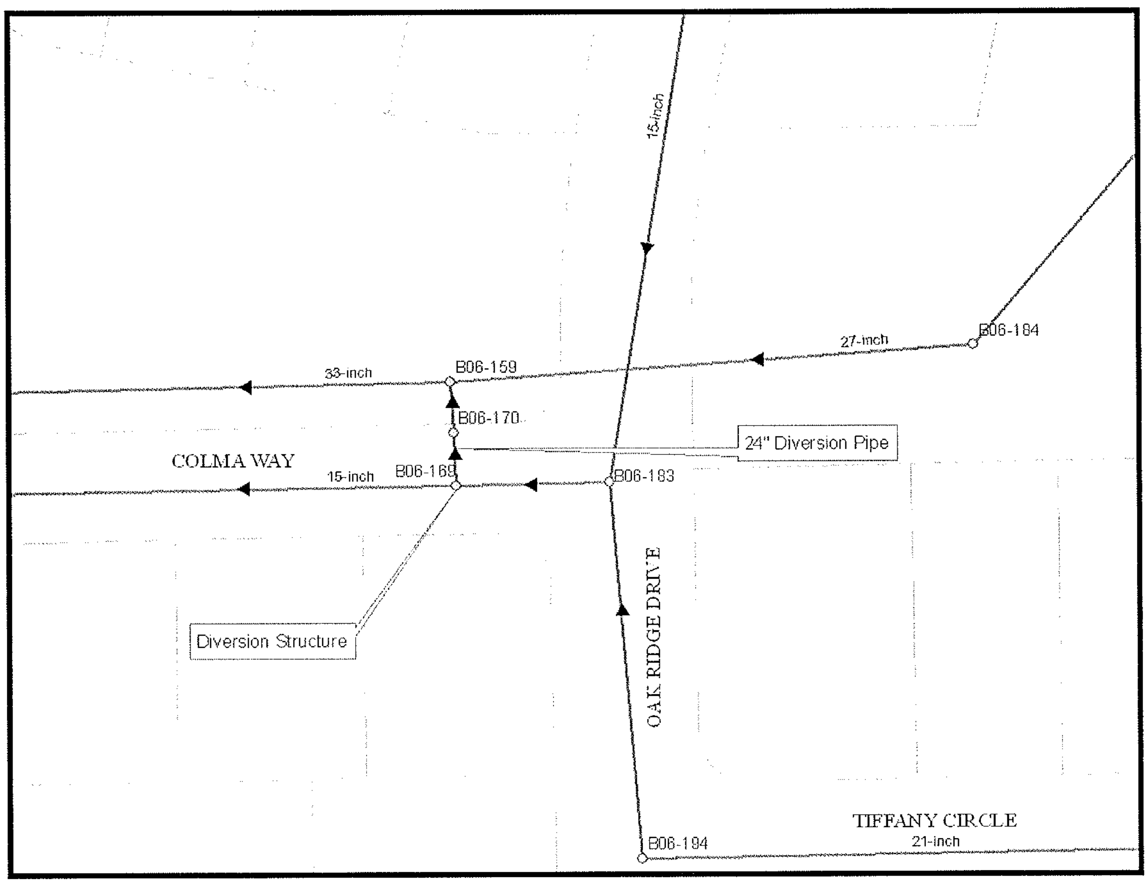


Figure 3 – Roseville Trunk Sewer Diversion at Manhole B06-169

2.4 Inverted Siphons

The model network has 8 inverted siphons which are shown on Figure 1. All inverted siphons have multiple barrels. Information on the individual barrels for each siphon was obtained the City of Roseville GIS. Flow was allocated to each barrel within H_2OMAP Sewer Pro based on the cross-sectional area of each barrel.

2.5 Pump Stations

The model of the existing trunk sewers includes three pump stations which are shown on Figure 1. Other pump stations within the City of Roseville and Placer County that are not included in the model are not located on trunk sewers that are 15 inches in diameter or larger. Information on the pump stations are presented in Table 2.

Johnson Ranch and Old Auburn pump stations are located in the City of Roseville. These two pump stations were designed to operate during peak wet weather flow events to transfer flow between trunk sewers and alleviate downstream capacity issues. These pump stations each have two pumps and currently operate in duty/standby mode with the standby pump operating only if the duty unit fails. It is feasible for these two pump stations to operate with both pumps running, providing additional peak wet weather pumping capacity. The City of Roseville staff has identified that these two pump stations can operate in a lead/lag mode because the pump stations are intended to reduce the peak wet weather flow events, and the pump stations are not needed to convey average dry weather flows. If a pump within the pump station were to fail, it could be replaced immediately after the peak wet weather flow event, when the pump station was shut down under normal operation. Table 2 presents the capacity of these two pump stations with either one or both pumps operating.

Dry Creek No. 1 pump station is located in Placer County, west of the Dry Creek WWTP. This pump station operates during dry and wet weather with three pumps in duty/standby mode. Table 2 presents the current and buildout capacity of this pump station with two pumps operating.

Table 2 – Pump Stations and Capacity Designations

Facility No.	Facility Name	Pumps	Duty/Standby Capacity[1] (mgd)	Lead/Lag Capacity[2] (mgd)
25	Johnson Ranch	2@1400 gpm	2.02	3.20
26	Old Auburn	2@300 gpm	0.43	0.68
NA	Dry Creek No. 1	3@1580 gpm	2.52[3]	NA

[1] Pump station capacity with one pump out of service
[2] Pump station capacity with both pumps operating
[3] Proposed pump station capacity at buildout with three 60 hp pumps (one standby) each rated at 1580 gpm. Pump station currently has a capacity of 1.73 mgd with two 20-hp duty pumps and one standby 60-hp pump.

2.6 Force Mains

Each pump station has an associated force main. Force main information is summarized in Table 3. The force mains are identified by their associated pump stations. The capacity of the force mains is based on a maximum flow velocity of 7 fps. The force main analysis does not include evaluation of hydraulic transients.

Table 3 – Force Mains

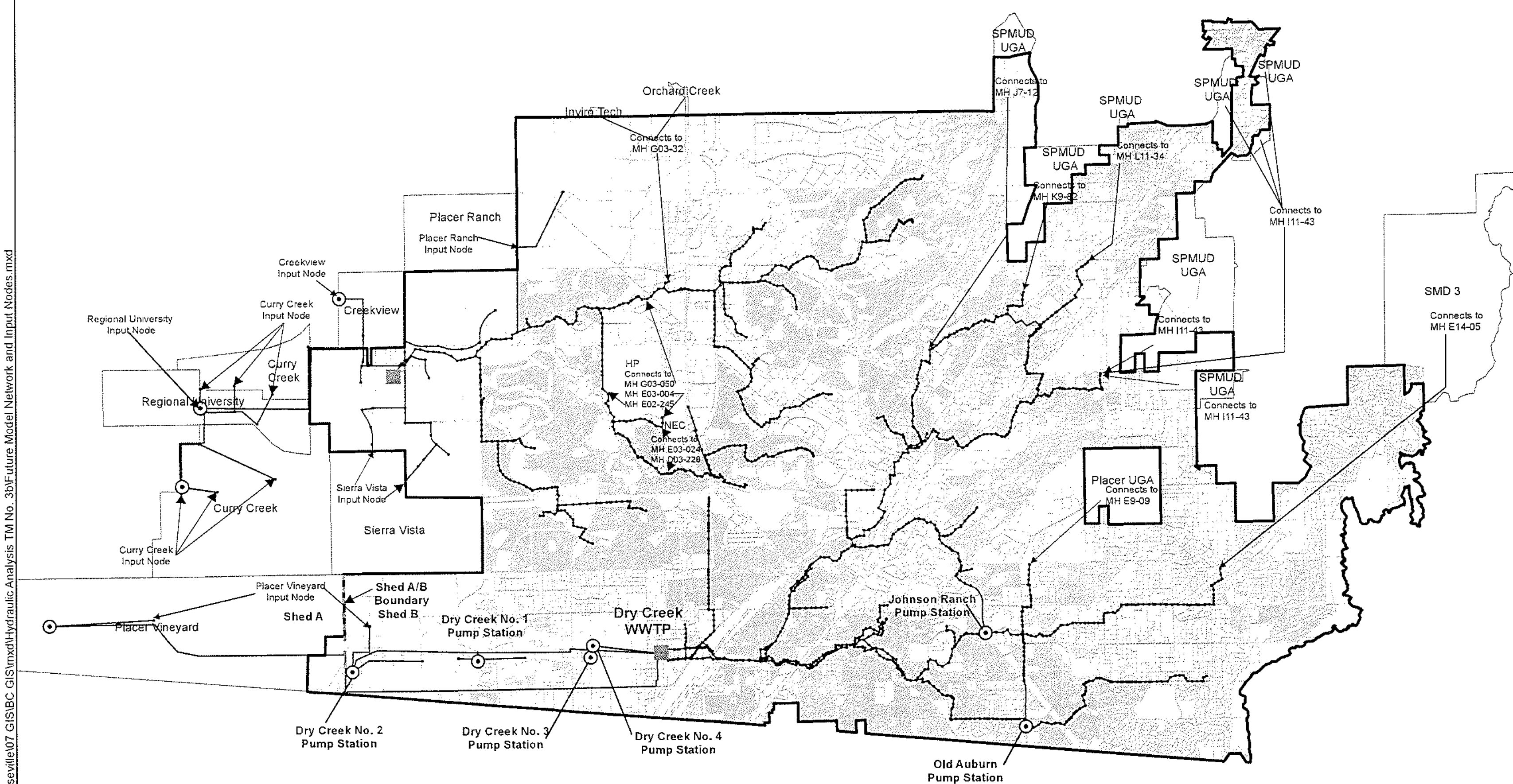
Facility No.	Facility Name	Diameter (in)	Length (ft)	Capacity (mgd)
25	Johnson Ranch	12	3,886	3.55
26	Old Auburn	8	3,358	1.58
NA	Dry Creek No. 1	16	14,100	6.31

2.7 Input Nodes

The model network contains input nodes at locations where flow is added to the model. Input nodes were located at each manhole in the model network. The amount of flow at the input nodes is based on the number of surrounding parcels and their land use designation. Parcels were generally assigned to the nearest pipe network input node using the Load Allocator feature within H_2OMAP Sewer Pro. The assignments were modified to account for the network of smaller diameter sewers that are not included in the model network. Each parcel in the model database has a corresponding input node associated with it.

Input nodes were also selected for future development areas, large point sources and UGAs that will contribute wastewater to the regional system. Most new developments, large point sources and UGAs in the eastern portion of regional service area were connected to the existing model network. New developments to the west of the 2005 Service Area boundary were connected to the proposed trunk sewers indicated in the future trunk sewer network. The input nodes that receive wastewater flow from these UGAs are indicated on Figure 4.

The HP and NEC point sources were loaded as indicated on Figure 4. For the buildout scenario at NEC, 1.0 mgd must be diverted from the NEC site south to the 30-inch trunk sewer along Foothills Road, south of Pleasant Grove Blvd. This will eliminate the potential for overloading the existing NEC sewer connection.



Legend

- Pump Station
- WWTP
- Modeled Manhole
- Modeled Pipe
- Parcel Boundary
- Urban Growth Areas

FIGURE 4

Future Model Network and UGA Input Nodes

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0.9 0.45 0 0.9
Scale in Miles

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3 Evaluation Criteria

This section summarizes the criteria for evaluating the capacity of the existing sewer system and for sizing relief sewer facilities, as potential capital projects. The criteria will be used to evaluate results of the hydraulic model.

3.1 Flows

The hydraulic performance assessment is performed using current and buildout wet weather flows. Peak wet weather flows include base sanitary flow, dry season and wet season groundwater infiltration (GWI) and rainfall dependent infiltration and inflow (RDI/I). A diurnal curve is applied to base sanitary flow to simulate the changes in base sanitary flow through a day. Base sanitary flows, defined in the Dry Weather Flow Projection TM (No. 2a), were applied in the model based on land use and the unit flow factors presented in Table 4.

Table 4 – Average Dry Weather Unit Flow Factors

Land Use Designation	Units	Trunk Sewer Analyses Unit Flow Factor[1]
Commercial	gpd per acre	800
Heavy Industrial	gpd per acre	800
Light Industrial	gpd per acre	800
Mixed Use	gpd per acre	2,160
Public/Quasi-Public	gpd per acre	620
Schools	gpd per acre	160
Residential 1 DU	gpd per DU	180
Residential 2 DU	gpd per DU	180
Residential 3 DU	gpd per DU	180
Residential Multiple DU[2]	gpd per acre	1,920
Open Space	gpd per acre	0
Parks > 10 Acres	gpd per acre	10
Vacant	gpd per acre	0

[1] Does not include an allowance for dry season GWI. Dry and wet season GWI are applied on an area-specific basis.
[2] The proposed Residential Multiple DU unit flow factor can also be represented as 130 gpd per DU.

Dry season GWI, defined in the Dry Weather Flow Projection TM (No. 2a), was applied spatially in the model to areas upstream of the permanent flow monitor sites as shown in Table 5. Dry season GWI was not applied to parks, open space, or Union Pacific Railroad property.

Table 5 – Dry Season GWI Rates

Basin	Tributary Area	Agency	GWI Rate
Pleasant Grove	Pleasant Grove WWTP Flow Monitor	Roseville	0 gpd/acre
Pleasant Grove	North Roseville Flow Monitor	SPMUD	0 gpd/acre
Pleasant Grove	Sunset Industrial Park	Placer County	0 gpd/acre
Dry Creek	Dry Creek WWTP Flow Monitor	Roseville	20 gpd/acre
Dry Creek	Springview Flow Monitor	SPMUD	210 gpd/acre
Dry Creek	Highlands Flow Monitor	SPMUD	0 gpd/acre
Dry Creek	Strap Ravine Flow Monitor	Placer County	120 gpd/acre
Dry Creek	Old Auburn Flow Monitor	Placer County	350 gpd/acre

Wet season GWI, defined in the Wet Weather Flow Projection TM (No. 2c), occurs in addition to the dry season GWI. Based on the results of this analysis, a wet season GWI rate of 200 gpd/acre was applied to developed parcels in the Dry Creek watershed. A wet season GWI rate of 100 gpd/acre was applied to developed parcels in the Pleasant Grove watershed. Wet season GWI was not applied to parks, open space, or Union Pacific Railroad property.

Design RDI/I is based on a 10-year 24-hour synthetic rainfall pattern that occurs across the entire service area. RDI/I flows are dependent on several factors including rainfall amount. RDI/I flows are typically projected using a design storm event. For this project, a 10-year, 24-hour design storm was chosen to project peak wet weather flows in the model. *Note: This is the design condition adopted by Sacramento County and recently required by the Central Valley Regional Water Quality Control Board in an order (official document adopted by the Board) to the City of Folsom.* The design storm hyetograph was developed utilizing Table 5-A-1 (elevation (h) = 150 feet) from the Placer County Flood Control and Water Conservation District Stormwater Management Manual (September 1, 1990). The peak rainfall hour was set at 6 a.m. so that the peak RDI/I response (which would normally occur about 1-2 hours after the rainfall for a typical basin) roughly coincides with the peak hour of the dry weather profiles to give a conservative flow response in the collection system. Further discussion on design flows is presented in the Wet Weather Flow Projection TM (No. 2c).

3.2 Pipe Network

Existing gravity sewers are evaluated primarily by the amount of surcharge that occurs within them under peak wet weather conditions. Hydraulic capacity, defined as the ratio of the peak flow in the pipe divided by capacity of the pipe, is used as a secondary evaluation criterion to develop relief sewer improvement projects. Additional pipe network evaluation criteria are provided below.

3.2.1 Surcharge

Gravity sewers are evaluated to identify areas with surcharge. Surcharge occurs when the hydraulic gradeline is above the crown of the pipe, indicating that the pipe would be flowing under pressure during surcharge conditions. The study area is anticipating significant growth in the future, so a conservative evaluation criterion of "no surcharging" was selected. Relief sewers would be considered as the potential capital project to eliminate surcharging under peak wet weather flow conditions.

The exception to this criterion is surcharging that is the result of "backwatering" from larger diameter pipes into smaller diameter pipes. Typically, at junctions between larger diameter and smaller diameter pipes, the crowns of the pipes are matched to avoid "backwatering". However, some pipe invert information that was used to develop the hydraulic model indicates that the crowns may not be matched at all junctions. In these cases, constructing relief sewers would not alleviate the "backwatering caused" surcharging.

3.2.2 Capacity

Existing sewers are also evaluated to identify hydraulic bottlenecks. Bottlenecks occur when the peak flow exceeds the calculated hydraulic capacity of an individual pipe reach. The modeling software program H_2OMAP Sewer Pro is used to determine the capacity of the existing and proposed gravity sewer lines. H_2OMAP Sewer Pro compares the calculated capacity of each pipeline to the peak flow and flags sewer reaches which have capacities that are less than the peak flows.

3.3 Pump Stations

Typically, pump stations need rated capacities that match or exceed the peak hourly wet weather flow (PWWF) for current and future conditions. The existing Old Auburn and Johnson Ranch pump stations divert some of the peak flow from the sewers where they are located. After discussions with the City, it was decided to allow these pump stations to be modeled to operate as peak wet weather pump stations with two pumps operating in a lead/lag mode (no standby pump). This criterion, rather than rating their capacity with one pump operating will be used to evaluate current and buildout PWWF projections. In contrast, the Dry Creek No. 1 pump station operates at all times and will be evaluated with the first and/or second pump operating and the third pump considered a standby pump that only operates if one of the duty pumps fails.

Additional detailed hydraulic assessments including surge analysis and field testing of actual pump capacities is beyond the scope of this System Evaluation.

3.4 Force Mains

The maximum velocity criterion for force mains is 7 feet per second (fps). Additional assessment of force mains, including surge analysis, is beyond the scope of this System Evaluation. The maximum velocity criterion is considered as an approximate indicator of the need to perform further assessment of a force main (if the criterion were exceeded under design conditions, for example).

4 Hydraulic Model Results

This section presents the results of the hydraulic modeling. The hydraulic evaluation of gravity sewers utilized the model network and the current and buildout PWWF projections developed for this study.

4.1 Current Gravity Sewer Evaluation

The results of the gravity sewer hydraulic analysis under ***current*** PWWF conditions are presented on Figure 5. The current land use scenario is defined in the Existing and Buildout Land Use TM (No. 1b) and consists of the parcels connected to the system as of June 2004. This figure identifies manholes in the trunk sewer network with surcharging and where the peak wet weather flows exceed the calculated capacities of the pipelines using Manning's equation in the model. Sewer profiles illustrating the hydraulic grade line at the time of PWWF are included in Attachment A.

Both hydraulic capacity deficiencies and backwater surcharging occurred under current PWWF conditions in a limited number of locations. Under current PWWF conditions, notable hydraulic capacity surcharging occurred in three general areas. Each of these areas is shown in Figures 5. Additional discussion on each of these pipe reaches is presented in the following sections.

The figures also show other surcharging and capacity issues that arise under current peak flow conditions but which are not included in the areas of concern discussed above. The surcharging is caused by backwatering and the capacity issues are localized and minimal. A number of these areas are immediately upstream of the inverted siphons. Gravity sewer improvements (or other substantial capital projects) are not needed to address these issues.

4.1.1 Area A – Placer County SMD-2

Area A is located upstream of the Old Auburn permanent flow monitoring site in Placer County. This 15-inch trunk sewer serves the southern portion of Granite Bay. Four of these pipe reaches surcharge for a period of approximately 1 hour during the current PWWF scenario.

4.1.2 Area D – Roseville

Area D is located in the Dry Creek WWTP basin in Roseville, east of Interstate 80. This 15-inch trunk sewer serves a portion of southeast Roseville and parallels the existing SPWA trunk sewer. There are no interconnects between the two sewers. Eleven of these pipe reaches surcharge for a period of approximately 4 hours during the current PWWF scenario. This surcharging can be eliminated if 0.9 mgd of PWWF is diverted to the adjacent 33-inch trunk sewer pipe. This diversion scenario is feasible as long as the Old Auburn pump station continues to divert PWWF from this trunk sewer network, which is planned for now and in the future.

4.1.3 Area E – Roseville

Area E is located in the Pleasant Grove WWTP basin in Roseville, along McAnally Road. This 15 and 18-inch trunk sewer serves a portion of western Roseville. Thirteen of these pipe reaches surcharge for a period of approximately 1 hour during the current PWWF scenario. Flow monitoring data indicated that this area had one of the highest rates of RDI/I in the SPWA service area. Modeling results indicated that this surcharging is solely attributable to the high I/I levels. The surcharging was eliminated when model run scenarios were completed with more typical I/I rates for this area. Roseville sewer operations staff indicated that a manhole may have been missing its cover in this area during the flow monitoring period and that the RDI/I rates may be inflated. City staff also commented that this area has not had historical hydraulic capacity issues. The City should perform an additional investigation in this area prior to considering the construction of relief sewers.

4.1.4 Area K – Roseville

Area K is located in the Dry Creek WWTP basin in Roseville. Area K has several inverted siphons and flat reaches of pipe that cause minor surcharging during current PWWF conditions. No improvement project is needed.

4.2 Buildout Gravity Sewer Evaluation

The results of the gravity sewer hydraulic analysis under ***buildout*** PWWF conditions are presented on Figure 6. The buildout land use scenario is defined in the Existing and Buildout Land Use TM (No. 1b). This figure identifies manholes in the trunk sewer network with surcharging and where the peak wet weather flows exceed the calculated capacities of the pipelines using Manning's equation in the model. Sewer profiles illustrating the hydraulic grade line at the time of PWWF are included in Attachment B.

Both hydraulic capacity deficiencies and backwater surcharging occurred under buildout PWWF conditions in a limited number of locations. Under buildout PWWF conditions, pipe reaches in 13 areas are projected to surcharge. Each of these areas is shown in Figure 6. Additional discussion on each of these pipe reaches is presented in the following sections.

The figures also show other surcharging and capacity issues that arise under current and future peak flow conditions but which are not included in the areas of concern discussed above. The surcharging is caused by backwatering and the capacity issues are localized and minimal. A number of these areas are immediately upstream of the inverted siphons. Gravity sewer improvements (or other substantial capital projects) are not needed to address these issues.

4.2.1 Area A – Placer County SMD-2 and Roseville

Area A is located upstream of the Old Auburn permanent flow monitoring site in Placer County. This 15-inch trunk sewer serves the southern portion of Granite Bay and the extreme southeast corner of Roseville. Thirteen pipe reaches in this area experience surcharging up to 3 feet for approximately 18 hours due to hydraulic capacity deficiencies for the buildout PWWF scenario.

4.2.2 Area B1 – Placer County SMD-2

Area B1 is located upstream of the Johnson Ranch pump station in Placer County. This 15 and 18-inch trunk sewer serves the northern portion of Granite Bay. A hydraulic analysis was performed in this area both with and without the SMD-3 UGA.

When a PWWF input of 1.85 mgd from the SMD-3 UGA is introduced into the model on this trunk sewer, fifty pipe reaches experience surcharging up to 4 feet for approximately 19 hours due to hydraulic capacity deficiencies. There are no hydraulic capacity deficiencies in this area if the SMD-3 UGA were not connected to the SPWA system for the buildout scenario. In a separate study, Placer County evaluated whether holding peak wet weather flows at the current SMD-3 WWTP would affect the SMD-2 trunk sewer. This study showed that a controlled release of 0.5 mgd from SMD-3 into SMD-2 would not adversely affect the trunk sewers in Area B1.

4.2.3 Area B2 – Roseville

Area B2 is located in Roseville upstream of the Johnson Ranch pump station and downstream of Area B1 and Area C. This 15 and 21-inch trunk sewer serves the northern portion of Granite Bay and a small area of Roseville. Nine pipe reaches in this area experience surcharging up to 11 feet for approximately 17 hours due to hydraulic capacity deficiencies for the buildout PWWF scenario. The surcharging in area B2 is caused by the proposed SMD-3 UGA and approximately 2700 acres of future development in Placer County and SPMUD that is loaded into the trunk sewer model upstream of Area C, which is tributary to Area B2. SPMUD has commented that some of this future development area may ultimately remain on septic tank service. For the buildout growth scenario (including SMD-3 UGA and approximately 2700 acres of future development in Placer County and SPMUD) a 24-inch replacement sewer is needed to resolve hydraulic capacity deficiencies. Without SMD-3, nine pipe reaches in this area experience surcharging up to 7 feet due to the future development in Placer County and SPMUD upstream of Area C and hydraulic relief is still needed. Without SMD-3 and the 2,700 acres of future development in Placer County and SPMUD, the two 15-inch pipe reaches (bottleneck downstream of the 18-inch and upstream of the 21-inch) surcharge less than two feet in pipes over 23 feet deep. These 15-inch pipe diameters are suspicious and should be investigated. However, no improvement project is needed for this deficiency.

4.2.4 Area C – Placer County SMD-2

Area C is tributary to Area B2 and is located along the Roseville City boundary. This 15-inch trunk sewer currently serves the northern portion of Granite Bay and a small area of Roseville (several parcels). Future development tributary to Area C includes the Placer UGA (very low density development) and approximately 2,700 acres of additional development within the 2005 Service Area in Placer County and SPMUD. This trunk sewer currently serves approximately 600 acres. SPMUD commented that some of this area may ultimately remain on septic tank service. Sixteen pipe reaches in this area experience surcharging up to 4 feet for approximately 18 hours due to hydraulic capacity deficiencies for the buildout PWWF scenario as a result of connections in Placer County and SPMUD. If these 2,700 acres of additional development are not loaded to the trunk sewer in Area C there is only minor surcharging (0.60 ft) for one flat pipe segment that is approximately 22 feet deep. No improvement project is needed for this deficiency.

4.2.5 Area D – Roseville

Area D is located in the Dry Creek WWTP basin in Roseville, east of Interstate 80. This 15-inch trunk sewer is a local Roseville sewer and only serves a portion of southeast Roseville. Nineteen pipe reaches in this area experience surcharging up to 5 feet for approximately 4 hours due to hydraulic capacity deficiencies for the buildout PWWF scenario. This surcharging can be eliminated if 1.1 mgd of PWWF is diverted to the adjacent 33-inch SPWA trunk sewer pipe. This diversion scenario is feasible as long as the Old Auburn pump station continues to divert PWWF from this trunk sewer network, which is currently planned.

4.2.6 Area E – Roseville

Area E is located in the Pleasant Grove WWTP basin in Roseville, along McAnally Road. This 15 and 18-inch trunk sewer serves a portion of western Roseville. Fourteen pipe reaches in this area experience surcharging up to 6 feet for approximately 2 hours due to hydraulic capacity deficiencies for the buildout PWWF scenario. Flow monitoring data indicated that this area had one of the highest rates of RDI/I in the SPWA service area. Modeling results indicated that this surcharging is solely attributable to the high I/I levels. The surcharging was eliminated when model runs were completed with more typical I/I rates for this part of Roseville. Also, Roseville sewer operations staff indicated that a manhole may have been missing its cover in this area during the flow monitoring period and that the RDI/I rates may be inflated. City staff also commented that this area has not had historical hydraulic capacity issues. The City should perform an additional investigation in this area prior to considering the construction of relief sewers.

4.2.7 Area F – Roseville

Area F is located in the Dry Creek WWTP basin in Roseville. This 15-inch trunk sewer serves a portion of Roseville and SPMUD. Five pipe reaches in this area experience minor surcharging during the buildout PWWF scenario with the hydraulic grade line less than two feet above the crown of the pipe (but 16 feet below grade). The hydraulic deficiencies are attributed four sections of flat pipe. The pipe is approximately 18 feet deep in the area of surcharge and there is no risk of overflow for the design PWWF. No improvement project is needed.

4.2.8 Area H1, H2, H3 and H4 – SPMUD

A significant number of pipe reaches in Areas H1, H2, H3 and H4 of SPMUD experience surcharging due to inadequate hydraulic capacity for the buildout PWWF scenario. Area H1 is an existing 15-inch trunk sewer that experiences up to six feet of surcharge for a period of 9 hours for the buildout PWWF. Area H2 is an existing 12 and 15-inch trunk sewer that experiences up to 13 feet of surcharge for a period of 20 hours for the buildout PWWF. Area H3 is an existing 24, 27 and 30-inch trunk sewer that experiences up to 3 feet of surcharge for a period of 13 hours for the buildout PWWF. Area H4 is an existing 18-inch trunk sewer that experiences up to 2 feet of surcharge for a period of 8 hours for the buildout PWWF.

According to SPMUD, these deficiencies are consistent with the results of their current wastewater collection system master plan. SPMUD will be identifying appropriate projects to relieve these sewer deficiencies separately.

4.2.9 Area I – Roseville

Area I is located in the Dry Creek WWTP basin in Roseville. Area I has very minor surcharging with the hydraulic gradeline at, or just above, the crown of the pipe. No improvement project is needed.

4.2.10 Area J – Placer County SMD-2

Area J is located in the Dry Creek WWTP basin in Placer County. Area J has very minor surcharging with the hydraulic gradeline at, or just above, the crown of the pipe. No improvement project is needed.

4.2.11 Area K – Roseville

Area K is located in the Dry Creek WWTP basin in Roseville. Area K has several inverted siphons and flat reaches of pipe that cause minor surcharging. No improvement project is needed.

4.2.12 Area L – Roseville

Area L is located in West Roseville. The proposed 18-inch and 24-inch pipe reaches from the intersection of Phillip Road and Westside Drive to the existing 36-inch stub at the PGWWTP influent junction structure are undersized for PWWF. The hydraulic deficiencies are attributed to the additional flow input into the West Roseville collection system from Creekview, Regional University, and Curry Creek UGAs. To carry the projected PWWF, these pipe reaches should be increased in size from 18-inches to 30-inches and 24-inches to 36-inches. The existing 36-inch stub out of the PGWWTP influent junction structure has is sufficiently sized to convey flow from the West Roseville Specific plan and Creekview, Regional University and Curry Creek UGAs.

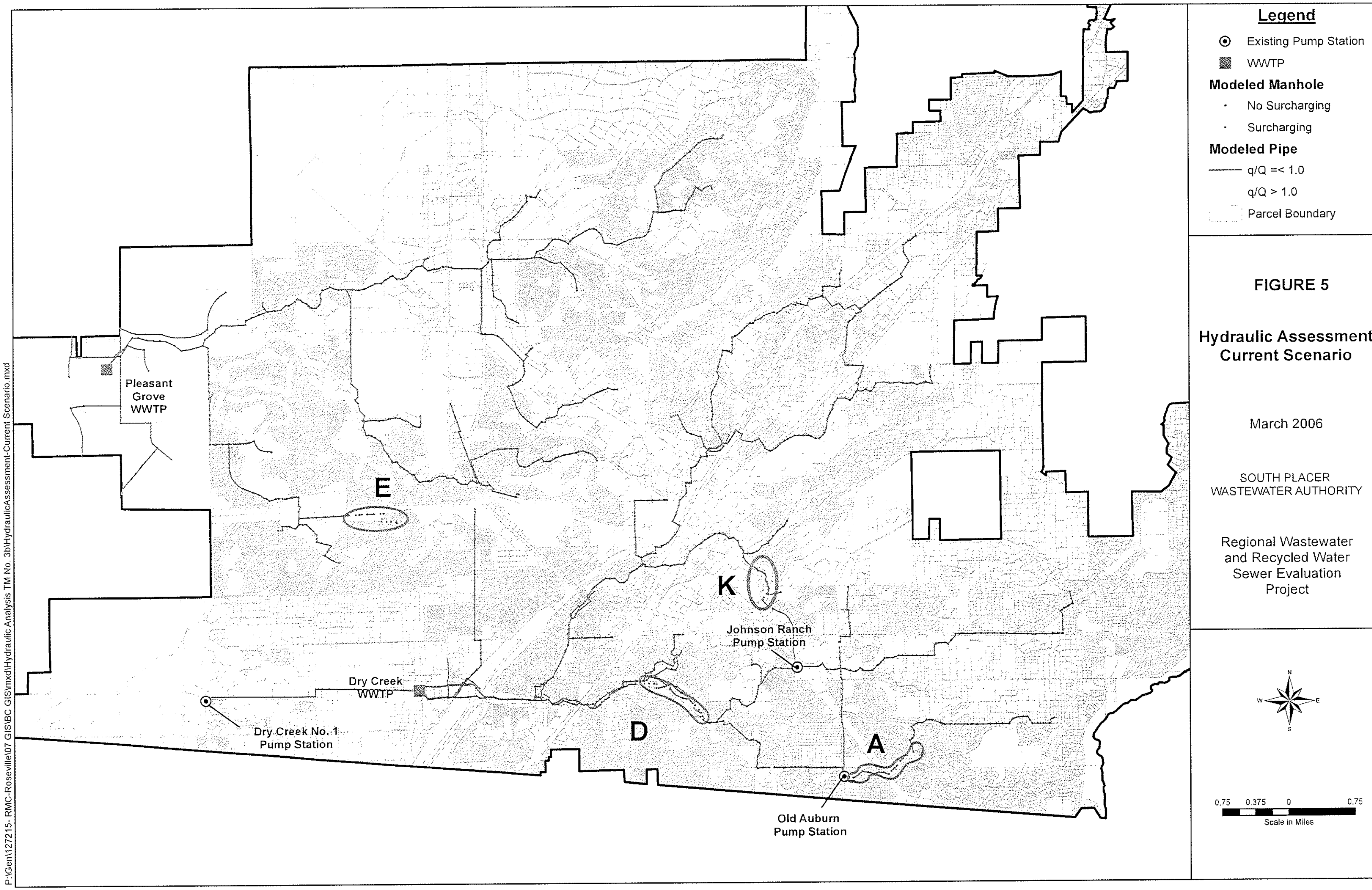

FIGURE 5

Hydraulic Assessment
Current Scenario

March 2006

SOUTH PLACER
WASTEWATER AUTHORITY

Regional Wastewater
and Recycled Water
Sewer Evaluation
Project

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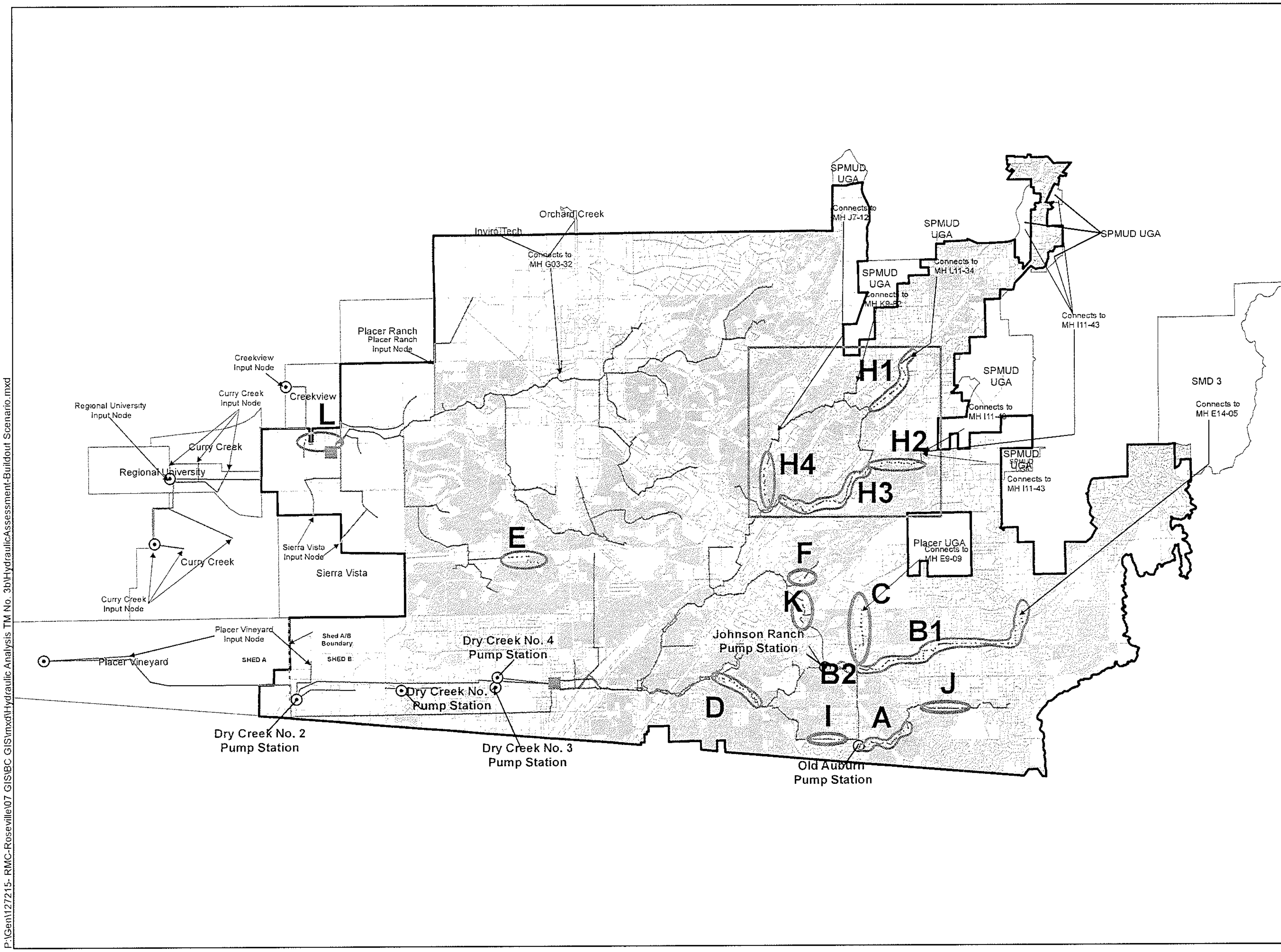


FIGURE 6

Hydraulic Assessment Buildout Scenario

March 2006

SOUTH PLACER WASTEWATER AUTHORITY

Regional Wastewater and Recycled Water Sewer Evaluation Project

4.3 Pump Station and Force Main Evaluation

The results of the pump station and force main hydraulic analysis under current and buildout PWWF conditions are presented in Table 6 and Table 7. Capacity issues with pump stations are determined by comparing the pump station capacities with the current and buildout PWWF. The capacities of the pump stations and force mains are based on information summarized in Table 2 and Table 3. All three pump stations and their associated force mains have capacity to meet the buildout PWWF. Buildout PWWF at Dry Creek No. 1 pump station is approximately 30 percent lower than values published in the Dry Creek West Placer Facilities Plan prepared by The Spink Corporation in November 1999. This difference is likely attributed to the new flow projection criteria utilized for this evaluation.

Table 6 – Pump Station Hydraulic Assessment

Facility No.	Facility Name	Duty/Standby (one pump) Capacity[1] (mgd)	Lead/Lag (two pump) Capacity[2] (mgd)	Current PWWF (mgd)	Buildout PWWF (mgd)
25	Johnson Ranch	2.02	3.20	0.00[4]	2.50[5]
26	Old Auburn	0.43	0.68	0.00[4]	0.68[5]
NA	Dry Creek No. 1	2.52[3]	NA	0.14[6]	1.77

[1] Pump station capacity with one pump not operating
[2] Pump station capacity with both pumps operating
[3] Proposed pump station capacity at buildout with three 60 hp pumps (one standby) each rated at 1580 gpm. Pump station currently has a capacity of 1.73 mgd with two 20-hp duty pumps and one standby 60-hp pump.
[4] Downstream trunk sewer can adequately convey current PWWF
[5] Buildout PWWF determined by identifying the amount of PWWF above the capacity of the downstream trunk sewers
[6] This is based on connected parcels in June 2004. Actual flow metering in October 2005 suggests Average Day Weather Flow may be as high as 0.13 mgd, which would translate to and estimated Current Peak Wet Weather flow of 0.3 mgd.

Table 7 – Force Main Hydraulic Assessment

Facility No.	Facility Name	Design Capacity (mgd)	Current PWWF (mgd)	Buildout PWWF (mgd)
25	Johnson Ranch	3.55	3.20	3.20
26	Old Auburn	1.58	0.68	0.68
NA	Dry Creek No. 1	6.31	1.73	3.29[1]

[1] Includes PWWF from the proposed Dry Creek Pump Station No. 2 which will share the common force main. Dry Creek Pump Station No. 2 service area includes Shed B of the Placer Vineyards Specific Plan service area. See Section 5.4 for further discussion.

4.3.1 Current PWWF Conditions

For current PWWF, the Old Auburn and Johnson Ranch pump stations have adequate capacity because the downstream trunk sewer systems have adequate capacity and flow is not diverted to the pump stations. The Dry Creek pump station has a current capacity of 1.73 mgd because it has not been expanded to serve buildout conditions. This is adequate to meet the current PWWF of 0.14 mgd. The force mains for all three pump stations have adequate capacity for current PWWF scenarios.

4.3.2 Buildout PWWF Conditions

For buildout PWWF (including the SMD-3 UGA), the Old Auburn and Johnson Ranch pump station capacities with one pump operating are inadequate. However, the pump stations have adequate capacity if operated with two pumps in lead/lag mode. The force mains also have adequate capacity for the current and buildout PWWF scenarios. Without the SMD-3 UGA, the Johnson Ranch pump station can operate with one pump and still meet the PWWF conditions.

The Dry Creek No. 1 pump station also has adequate capacity to meet the buildout PWWF scenario. However, this pump station may need to be upgraded as outlined in the Dry Creek facility plan to meet flow projections for the buildout condition. It is possible that these pumps may not need to be as large as originally planned. The buildout PWWF for the Dry Creek No. 1 pump station is 1.77 mgd versus the design capacity of 2.52 mgd. The force mains for all three pump stations (Old Auburn, Johnson Ranch and Dry Creek No. 1) have adequate capacity for the buildout PWWF scenarios. It appears that the existing 16-inch force main serving Dry Creek No. 1 pump station may have enough capacity (without exceeding the 7 fps maximum velocity criteria) to serve all four pump stations that will eventually serve the West Dry Creek service area. However, due to the complex hydraulics associated with four pump stations sharing a common force main, a detailed hydraulic analysis should be performed prior to selecting this alternative.

5 Improvement Projects

This section describes the criteria used for developing and pricing hydraulic capacity improvement projects in the regional and regional partners' collection system. Thirteen projects have been identified to (1) address hydraulic deficiencies by potential improvements to existing facilities or by diverting flow, and (2) to extend service to new development.

5.1 Criteria

Criteria were identified for developing potential improvements to the collection system to accommodate current and future flows.

5.1.1 New Replacement Sewers

New replacement sewers are provided to increase hydraulic capacity and to eliminate capacity-related surcharging. New sewers are sized to replace the existing sewer with a larger diameter sewer.

5.1.2 Minimum size

New replacement sewers will be sized so that the peak hourly flow rate in the replacement sewer will not exceed the full pipe capacity.

5.1.3 Slope

New replacement sewers are developed using the slope of the existing sewer or associated sewers. The slopes of the new sewers are constrained by the upstream and downstream invert elevations of the existing sewers. The flow velocities in the new sewers may be less than typical design standards due to the constraint of the existing invert elevations.

5.1.4 Pump Stations

New pump stations have been sized with one or more duty pumps and one pump operating in a standby mode (as opposed to the wet weather peaking pump stations). Duty/standby pump station operations are typical for wastewater pumping stations to prevent a sewer overflow in the event that the duty pump fails. The determination of the number of duty pumps will be made during the project design phase (subsequent to the system evaluation).

The existing Old Auburn and Johnson Ranch pump stations only operate during peak flow events and only to divert some of the peak flow from where they are located. After discussions with the City staff, it was decided to identify the operating criteria for these pump stations so that they operate as peak wet weather pump stations with capacity defined as "both pumps in a lead/lag mode (no standby pump)". These criteria will be used to determine if capacity improvements are required.

5.2 Capital and Construction Costs

Capital and construction costs presented in this TM represent preliminary cost estimates of the costs to plan and engineer projects, and the materials, labor and services necessary to build the proposed projects. The cost estimates are indicative of the cost of construction in the study area. In considering cost estimates, it is important to realize that changes during final design, as well as future changes in the cost of material, labor and equipment, will cause comparable changes in the estimated costs. Construction cost data given in this report is not intended to represent the lowest prices that can be achieved, but rather it is intended to represent planning-level estimates for budgeting purposes.

The unit capital costs for gravity sewer and force main pipeline construction were developed based on the sewer pipeline replacement costs shown in Tables 2-3 and 2-6 of the City of Roseville Infrastructure Rehabilitation Plan (April 2003). These costs were developed from recent projects in the City of Roseville and include allowances for engineering and administration (including construction management). The unit capital costs have been adjusted by approximately 8 percent for increasing construction cost considerations since April 2003. Costs for construction of new large diameter sewers would significantly increase if extensive utility relocation and traffic control were required. Pipeline unit capital costs are presented in Table 8.

Pump station capital and construction costs are based on cost curves from Pumping Station Design, Second Edition by Robert L. Sanks. This reference book is an industry standard for pump station design. There is no capital cost associated with changing operating modes/capacity definitions of the Old Auburn and Johnson Ranch pump stations.

Capital costs were increased by 30 percent to account for contingencies. A contingency allowance is appropriate given the planning level of the capital cost estimates and provides a conservative cost estimate that is suitable for budgeting purposes.

Table 8 –Pipe Unit Capital Costs (ENR CCI Value = 8435)

Pipe Diameter (in)	Conveyance	Pipe Material[1]	Replacement Cost[1] $/ft	Replacement Cost[1] $/dia-inch
8	Gravity	VCP	164	21
8	Force Main	PVC	120	15
10	Gravity	VCP	205	21
10	Force Main	PVC	150	15
12	Gravity	VCP	246	21
15	Gravity	VCP	292	19
16	Force Main	PVC	224	14
18	Gravity	VCP	330	18
20	Force Main	PVC	260	13
21	Gravity	VCP	363	17
24	Gravity	VCP	415	17
24	Force Main	PVC	312	13
27	Gravity	VCP	437	16
30	Gravity	VCP	454	15
30	Force Main	PVC	360	12
33	Gravity	VCP	463	14
36	Gravity	VCP	505	14
42	Gravity	VCP	544	13
48	Gravity	RCP	622	13
60	Gravity	RCP	713	12
66	Gravity	RCP	713	11
72	Gravity	RCP	778	11

[1] Includes allowance for engineering and administration (including construction management)

5.3 Projects for Existing Facilities (Part 1)

The proposed improvements to the existing collection system are shown on Figure 7. Improvements were developed to address the hydraulic deficiencies discussed above and to prevent capacity-related surcharging in the improved collection system. *This Section (Part 1) does not include projects that are attributed to the SMD-3 UGA and approximately 2,700 acres of future development in Placer County and SPMUD within the 2005 Service Area that is tributary to Area C.*

The improvement projects and their estimated CIP Budgetary Cost estimates (with a 30 percent contingency included) are summarized in Table 9 and Attachment C. Prior to constructing these projects, flow monitoring and other site specific investigations should be conducted on the critical line segments to validate and refine the model results. *The entity with Primary Responsibility for the specific improvement project is indicated in Table 9 and the project headings listed below.* "Placer County" refers to development within Placer County SMD-2.

5.3.1 Improvement Project 1 – Area A (Primary Responsibility: Placer County)

An 18-inch replacement sewer is needed to improve the hydraulic deficiencies identified in Area A through the hydraulic modeling process. Redirecting flow to another trunk sewer is not feasible. This project extends 5,000 feet from manhole B11-16 to A08-156. This project is located in an existing sewer easement between Roseville Parkway and Sierra College Boulevard and a new alignment may be necessary. To conservatively estimate the cost of this new alignment without undertaking site specific investigations, the length of this project was increased by 50 percent over the current deficient length to develop a higher cost estimating allowance. The determination of the new alignment will be made during design.

5.3.2 Improvement Project 5 – Area D (Primary Responsibility: Roseville)

A diversion manhole structure is needed to improve the hydraulic deficiencies identified in Area D. This structure should divert approximately 1.1 mgd of PWWF at manhole B06-313 to the existing 33-inch SPWA trunk sewer. This project is located near the intersection of Oak Ridge Drive and Caloma Way in an existing sewer easement along Linda Creek. This diversion structure will improve the hydraulics in this area because, and only because the Old Auburn Pump Station is planned by the City to operate as specified in Section 4, with corresponding capacity ratings.

5.3.3 Improvement Project 6 – Area E (Primary Responsibility: Roseville)

An additional investigation is needed in Area E prior to the construction of any replacement sewer. This project extends 4,000 feet from manhole D03-100 to D02-353. This project is located in the McAnally Road street right-of way and a sewer easement south of Pleasant Grove Road. Additional investigation is needed due to the uncertainty in the flow monitoring data at this site. This was previously discussed in the current and buildout gravity sewer evaluation (Section 4). The additional investigation should include the following items:

- Flow monitoring during the wet season at Basin 7 as identified in the Wet Weather Flow Projection TM (No. 3b).
- Elevation survey of approximately 18 manhole inverts between manholes D03-100 to D02-353 to confirm pipe slopes.
- Visual surcharge checks of the pipes in question during heavy rainfall.

RDI/I reduction to levels seen in adjacent sewer basins would also eliminate the need for this improvement project. As a last resort, if the additional investigations did not eliminate the basis of the hydraulic deficiencies identified by the modeling, a 21-inch replacement sewer would be needed to improve the hydraulic deficiencies identified in Area E. For contingency planning purposes, the cost for this project is included in Table 8.

5.3.4 Improvement Project 7 – Area L (Primary Responsibility: Regional University UGA and Curry Creek UGA)

This project extends from the intersection of Phillip Road and Westside Drive to the existing 36-inch pipe stub that connects with the influent junction structure at Pleasant Grove WWTP. The deficient sewers in Area L have been designed and will be constructed in early 2006. The deficiencies identified in the model can be corrected if 30-inch and 36-inch pipe is constructed in lieu of the planned 18-inch and 24-inch trunk sewer. Another option is to route the Regional University force main directly to the 36-inch pipe that connects with the influent junction structure. This 36-inch stub is sufficiently sized to convey PWWF from the West Roseville Specific Plan area and Creekview, Regional University and Curry Creek UGAs.

5.3.5 Improvement Project 8 – Area H1, H2, H3 and H4 (Primary Responsibility: SPMUD)

Improvement projects in SPMUD have not been developed at the request of SPMUD and a cost is not provided in Table 8. SPMUD will be identifying appropriate projects to relieve these sewer deficiencies separately.

Table 9 – Existing Facility Project Summary (Part 1)[4]

Project No.	Primary Responsibility	Item	Quantity	Unit Cost ($)	Estimated Capital Cost ($)	Proposed CIP Budget Cost[1] ($)
1-Area A	Placer County	18-inch Gravity	7,500[2] lf	330	2,475,000	3,218,000
Project 1 Subtotal					**2,475,000**	**3,218,000**
5- Area D	Roseville	Diversion Structure	1 ea	30,000	30,000	39,000
Project 5 Subtotal					**30,000**	**39,000**
6- Area E[3]	Roseville	21-inch Gravity	4,000 lf	363	1,452,000	1,888,000
Project 6 Subtotal					**1,452,000**	**1,888,000**
7- Area L	Regional Univ. UGA Curry Creek UGA	30-inch Gravity	1,500 lf	454	681,000	885,000
		36-inch Gravity	3,000 lf	505	1,515,000	1,970,000
Project 7 Subtotal					**2,196,000**	**2,855,000**
Total Cost					**6,153,000**	**8,000,000**

[1] Includes 30 percent contingency
[2] Includes 50 percent allowance for alternative alignment
[3] This project needed only if additional investigation identifies it as a necessary project
[3] This Table does not include Projects 2, 3 and 4 which are solely attributed to the SMD-3 UGA and approximately 2,700 acres of future development in Placer County and SPMUD within the 2005 Service Area. These projects are included in Section 5.4.

5.4 Projects for Existing Facilities (Part 2)

The proposed improvements to the existing collection system are shown on Figure 7. Improvements were developed to address the hydraulic deficiencies discussed above and to prevent capacity-related surcharging in the improved collection system. *This Section (Part 2) only includes projects that are attributed to the SMD-3 UGA and approximately 2,700 acres of future development in Placer County and SPMUD within the 2005 Service Area that is tributary to Area C.*

The improvement projects and their estimated CIP Budgetary Cost estimates (with a 30 percent contingency included) are summarized in Table 10 and Attachment C. Prior to constructing these projects, flow monitoring and other site specific investigations should be conducted on the critical line segments to validate and refine the model results. *The entity with Primary Responsibility for the specific improvement project is indicated in Table 10 and the project headings listed below.* "Placer County" refers to development within Placer County SMD-2.

5.4.1 Improvement Project 2 – Area B1 (Primary Responsibility: SMD-3 UGA)

Based on the model results discussed in Section 4, a 21-inch replacement sewer is needed to improve the hydraulic deficiencies identified in Area B1. Redirecting flow to another trunk sewer is not feasible. These deficiencies are solely attributed to the SMD-3 UGA and this project is not needed if the flow from SMD-3 was limited to 0.5 mgd (storage scenario).

Area B1 extends 18,000 feet from manhole E14-05 to B08-042. This project is located upstream of the Johnson Ranch Pump Station in an existing sewer easement and in the Douglas Boulevard street right-of way; a new alignment may be necessary. To estimate the cost of this new alignment, the length of this project was increased by 50 percent over the current deficient length to develop a higher cost estimating allowance. The determination of the new alignment will be made during design.

5.4.2 Improvement Project 3 – Area B2 (Primary Responsibility: Placer County, SMD-3 UGA and SPMUD)

A 24-inch replacement sewer is needed to improve the hydraulic deficiencies identified in Area B2. Redirecting flow to another trunk sewer is not feasible. These deficiencies are attributed to the SMD-3 UGA and approximately 2,700 acres of future development in Placer County and SPMUD within the 2005 Service Area. This project is not needed if flows from SMD-3 and the future growth areas within Placer County and SPMUD (upstream of Area C) were directed elsewhere.

Area B2 extends 3,000 feet from manhole B08-042 to B07-405. This project is located upstream of the Johnson Ranch Pump Station in an existing sewer easement and a new alignment may be necessary. To estimate the cost of this new alignment, the length of this project was increased by 50 percent over the current deficient length to develop a higher cost estimating allowance. The determination of the new alignment will be made during design.

5.4.3 Improvement Project 4 – Area C (Primary Responsibility: Placer County and SPMUD)

A 21-inch replacement sewer is needed to improve the hydraulic deficiencies identified in Area C. Redirecting flow to another trunk sewer is not feasible. These deficiencies are attributed to 2,700 acres of future development in Placer County and SPMUD within the 2005 Service Area. This project is not needed if flow from the 2,700 acres of future development in Placer County and SPMUD within the 2005 Service Area was directed elsewhere. This project extends 6,000 feet from manhole E9-09 to B08-042. This project is primarily located in the Sierra College Boulevard and Cavitt Stallman Road street right-of way. It extends from the Strap Ravine trunk sewer north to Olive Ranch Road.

Table 10 – Existing Facility Project Summary (Part 2)

Project No.	Primary Responsibility	Item	Quantity	Unit Cost ($)	Estimated Capital Cost ($)	Proposed CIP Budget Cost[1] ($)
2- Area B1	SMD-3 UGA	21-inch Gravity	27,000[2] lf	363	9,801,000	12,741,000
Project 2 Subtotal					**9,801,000**	**12,741,000**
3- Area B2	Placer County SMD-3 UGA SPMUD	24-inch Gravity	4,500[2] lf	415	1,868,000	2,428,000
Project 3 Subtotal					**1,868,000**	**2,428,000**
4- Area C	Placer County SPMUD	21-inch Gravity	6,000 lf	363	2,178,000	2,831,000
Project 4 Subtotal					**2,178,000**	**2,831,000**
Total Cost					**13,847,000**	**18,000,000**

[1] Includes 30 percent contingency
[2] Includes 50 percent allowance for alternative alignment

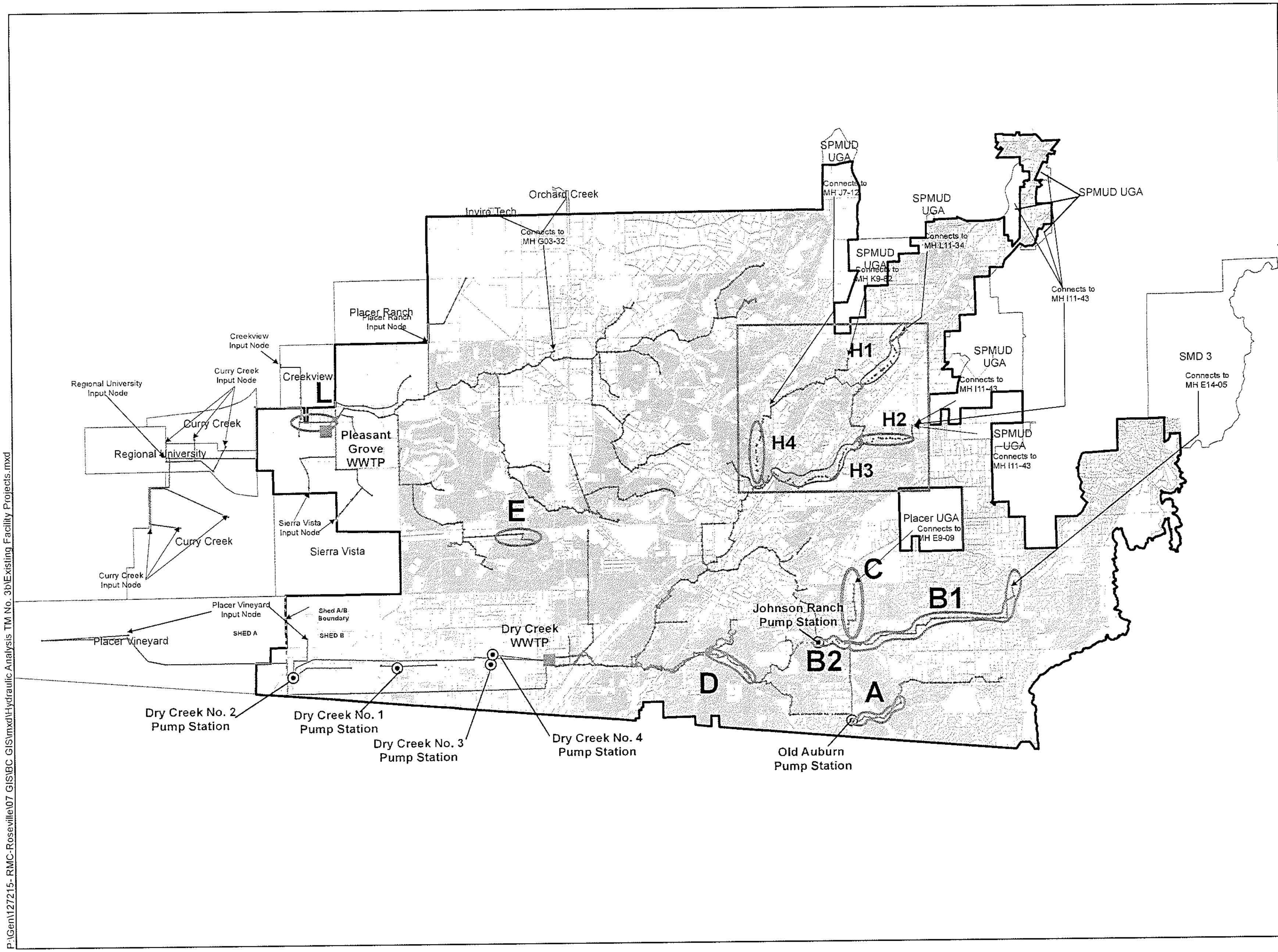


FIGURE 7

Existing Facility Projects

March 2006

SOUTH PLACER WASTEWATER AUTHORITY

Regional Wastewater and Recycled Water Sewer Evaluation Project

5.5 Projects to Extend Service

The proposed improvements needed to extend service to the proposed Urban Growth Areas west of Roseville are shown on Figure 8. Service extension projects are identified by the UGA they serve in Table 11. Project information is current based on information provided by UGA applicants at the time of this evaluation and changes may evolve over time. The proposed Total Project Costs identified in Table 10 are for informational purposes only. These costs have been developed utilizing a unit cost table representative of municipal sewer projects in the SPWA area. Actual sewer infrastructure costs for each extension project will be the responsibility of the developer.

5.5.1 Extension Project 1 – Placer Ranch

The proposed improvements to extend service into Placer Ranch were identified in the Placer Ranch Specific Plan and were included in the trunk sewer model. Flow from Placer Ranch and some areas of Placer County north of Placer Ranch are introduced into the existing trunk sewer that is tributary to the Pleasant Grove WWTP. Proposed pipe diameters are included in Figure 8 and Table 11.

5.5.2 Extension Project 2 – Placer Vineyards and West Dry Creek

The proposed improvements to extend service into Placer Vineyards and West Dry Creek were identified in the Placer Vineyards Specific Plan and the West Dry Creek Facilities Plan and were included in the trunk sewer model. The West Dry Creek service area is located between the Dry Creek WWTP and the Placer Vineyards UGA. The Placer Vineyards UGA includes one pump station. This pump station will roughly serve the area of Placer Vineyards outside the 2005 Service Area (Shed A). This pump station will pump directly to the Dry Creek WWTP. The area of Placer Vineyards roughly within the 2005 Service Area (Shed B) will be served by the proposed Dry Creek Pump Station No. 2. Dry Creek Pump Station No. 2 will pump into a common force main already serving the existing Dry Creek Pump Station No. 1 which flows directly to the Dry Creek WWTP. Two additional pump stations (No. 3 and No. 4) to serve the eastern portion of West Dry Creek are also proposed and will share a common force main to the Dry Creek WWTP. Proposed pipe diameters and pump station capacities for the Placer Vineyards UGA and West Dry Creek are included in Table 11 and Figure 8.

5.5.3 Extension Project 3 – Regional University

The proposed improvements to extend service into the Regional University UGA were identified in the Regional University Specific Plan and were included in the trunk sewer model. The Regional University UGA includes one pump station and is able to collect flow from Curry Creek North and South. Flow from the Regional University pump station is pumped (along with flow from Curry Creek South) east through Regional University and north along Watt Avenue to a gravity sewer main on Phillip Road. This gravity main flows east to the West Roseville collection system, tying into the proposed 18-inch gravity sewer at Westside Drive and Phillip Road transitioning to a 24-inch as the proposed gravity sewer heads east to join the existing Philip Road. Note that for this gravity sewer option to provide service to Regional University and Curry Creek, the gravity sewers planned for West Roseville (the 18-inch/24-inch) would have to be upsized to 42-inch and 48-inch, respectively. An alternate alignment is to pump directly from Regional University to the 36-inch gravity sewer that ties into the influent junction structure at Pleasant Grove WWTP. The proposed pipe diameters and pump station capacities are included in Figure 8 and Table 11.

5.5.4 Extension Project 4 – Curry Creek

The proposed improvements to extend service into Curry Creek were identified in the Regional University Master Sewer Study. Pipes in Curry Creek North will flow by gravity into Regional University. Flow from Curry Creek South will be pumped north where the Curry Creek South force main will tee into the proposed Regional University force main. An alternate alignment for the Curry Creek South force main is to go east across Curry Creek and then north at Watt Avenue to a junction point on the Regional University force main. The proposed pipe diameters and lift station capacity are included in Figure 8 and Table 11.

5.5.5 Extension Project 5 – Creekview

The proposed improvements to extend service into Creekview were not previously identified. USGS topographic contour information indicates that a pump station will be necessary to transport flow into the West Roseville collection system. The proposed pipe diameters and pump station capacity are included in Figure 8 and Table 11.

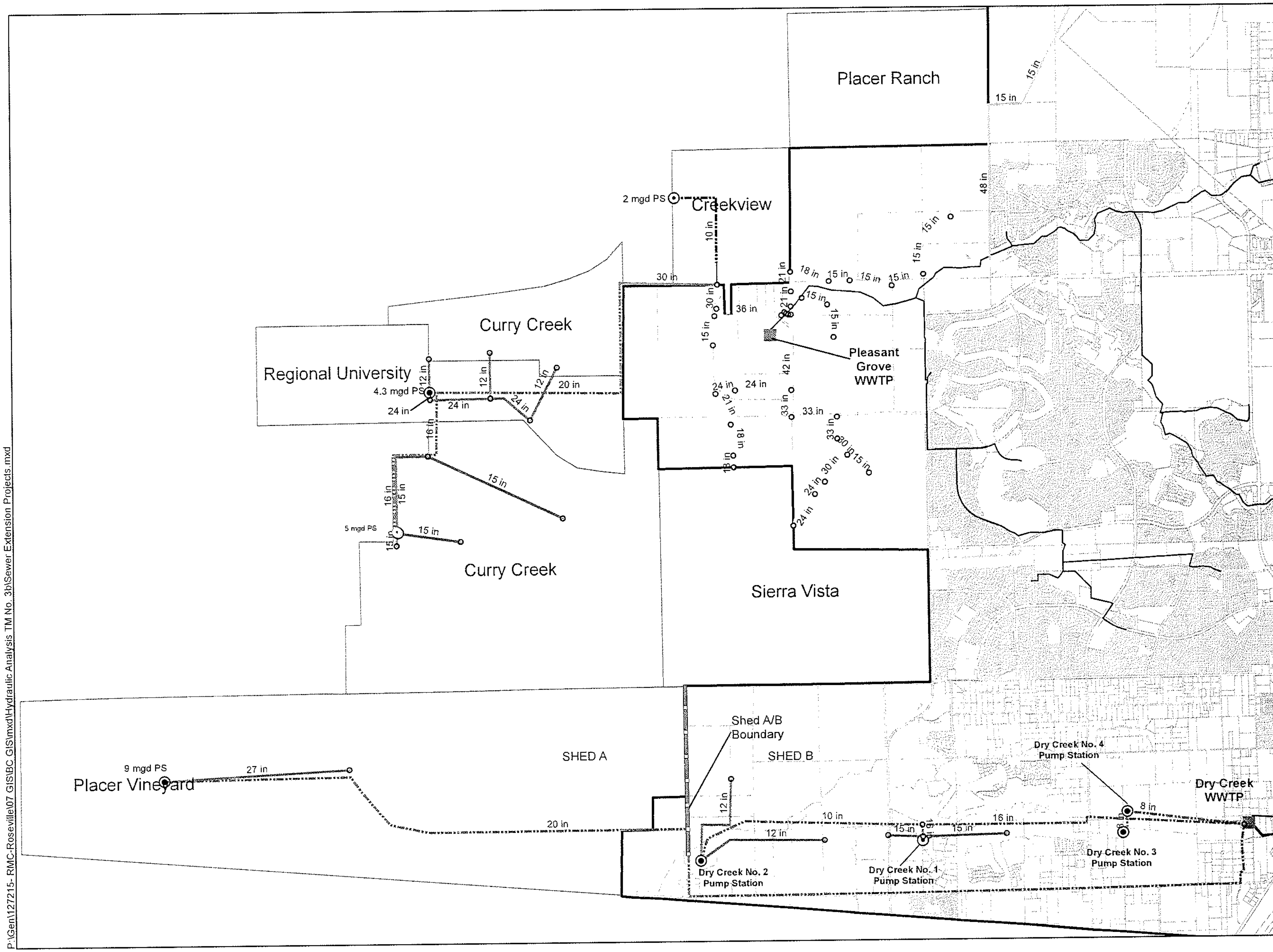
5.5.6 Extension Project 6 – Sierra Vista

The proposed improvements to extend service into Sierra Vista were not previously identified. Flows from Sierra Vista will tie into two existing sewer stubs along the border of Sierra Vista and West Roseville. Approximately 1/3 of Sierra Vista is projected in our modeling to be served to the 18-inch trunk sewer in West Roseville and 2/3 of the area is projected to be served to the 24-inch trunk sewer in West Roseville. Most of the Sierra Vista Service Area can be served to the existing West Roseville sewer stubs by gravity. A small area in the southwest portion of Sierra Vista may require a local pump station. A trunk sewer network and corresponding sewer extension project was not established for the Sierra Vista service area.

Table 11 – Sewer Extension Project Summary[1]

Project No.	Item	Quantity	Unit Cost ($)	Estimated Capital Cost ($)	Proposed Total Project Cost[2] ($)
1-Placer Ranch	48-inch gravity	6,000 lf	622	3,732,000	4,852,000
	15-inch gravity	5,700 lf	292	1,664,000	2,164,000
Project 1 Subtotal				**5,396,000**	**7,016,000**
2-Placer Vineyards/ West Dry Creek	12-inch gravity - PV	4,400 lf	246	1,082,000	1,407,000
	27-inch gravity - PV	7,300 lf	437	3,190,000	4,147,000
	20-inch force main - PV	48,000 lf	260	12,480,000	16,224,000
	9 mgd pump station - PV	1 ea	3,000,000	3,000,000	3,900,000
	12-inch gravity - DC	5,100 lf	246	1,255,000	1,631,000
	15-inch gravity - DC	4,700 lf	292	1,372,000	1,784,000
	10-inch force main – DC No. 2	9,500 lf	150	1,425,000	1,853,000
	1.5 mgd pump station – DC No. 2	1 ea	600,000	600,000	780,000
	0.6 mgd pump station – DC No. 3	1 ea	400,000	400,000	520,000
	1.2 mgd pump station – DC No. 4	1 ea	600,000	600,000	780,000
	8-inch force main – DC No. 3 and 4	5,600 lf	120	672,000	874,000
Project 2 Subtotal				**26,076,000**	**33,920,000**
3-Regional University	24-inch gravity	5,000 lf	415	2,075,000	2,698,000
	30-inch gravity	2,900 lf	454	1,317,000	1,712,000
	20-inch force main	12,000 lf	260	3,120,000	4,056,000
	4.4 mgd pump station	1 ea	2,000,000	2,000,000	2,600,000
Project 3 Subtotal				**8,512,000**	**11,066,000**
4-Curry Creek	12-inch gravity	6,000 lf	246	1,476,000	1,919,000
	15-inch gravity	13,000 lf	292	3,796,000	4,935,000
	16-inch force main	7,000 lf	224	1,568,000	2,038,000
	5 mgd lift station	1 ea	2,000,000	2,000,000	2,600,000
Project 4 Subtotal				**8,840,000**	**11,492,000**
5-Creekview	10-inch force main	4,100 lf	150	615,000	800,000
	2 mgd pump station	1	1,500,000	1,500,000	1,950,000
Project 5 Subtotal				**2,115,000**	**2,750,000**
Total Cost				**50,939,000**	**66,244,000**

[1] Proposed Sewer Extension Project Cost Table is for informational purposes only. These costs have been developed utilizing a unit cost table representative of municipal sewer projects in the SPWA area. Actual sewer infrastructure costs for each extension project will be the responsibility of the developer.
[2] Includes 30 percent contingency



Legend

- Pump Station
- WWTP
- Modeled Manhole
- Existing Pipe
- Force Main

Extension Project

- West Roseville
- Placer Ranch
- Placer Vineyard
- Regional University
- Curry Creek
- Creekview
- Parcel Boundary

FIGURE 8

Sewer Extension Projects

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Regional Wastewater and Recycled Water Sewer Evaluation Project

3,000 1,500 0 3,000
Feet

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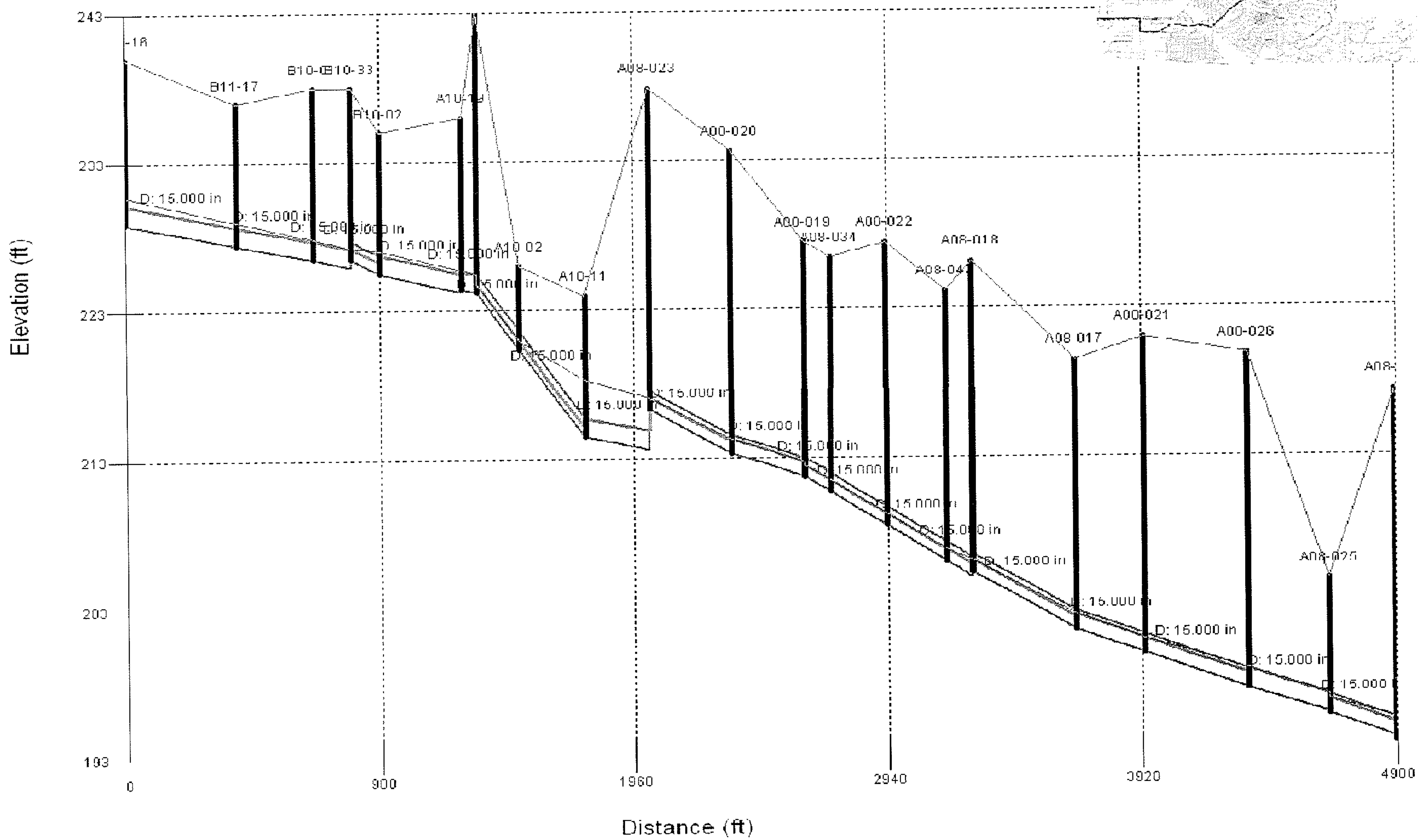
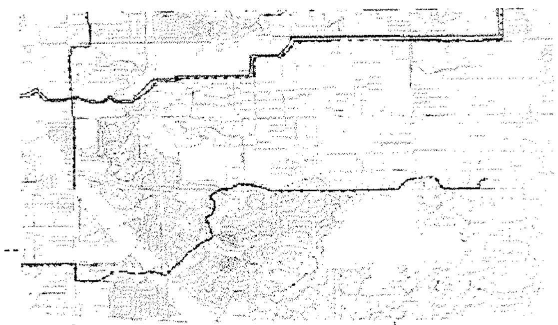
Attachment A

Sewer Profiles – Current Scenario

Current Model Results

Project Area A

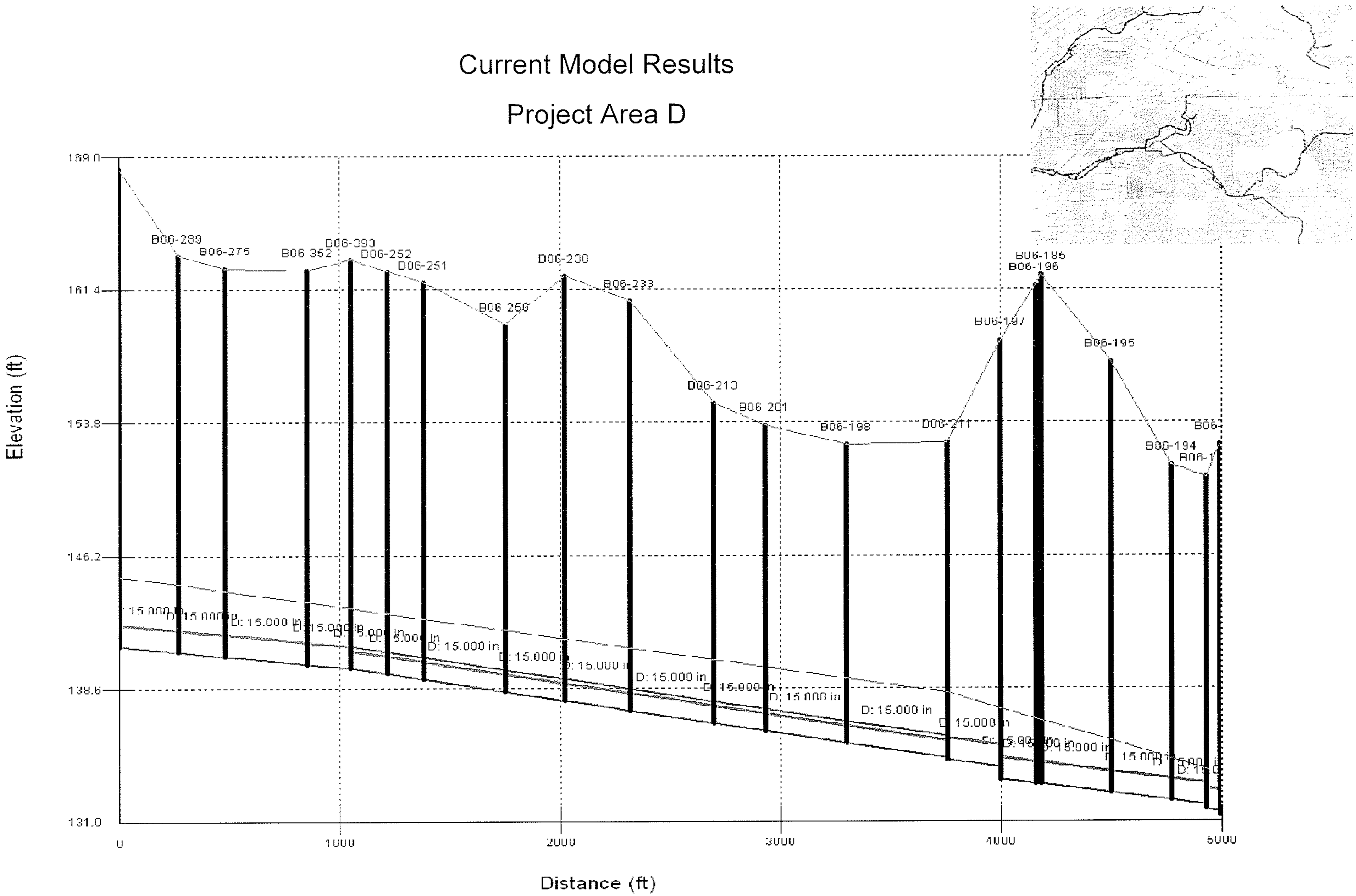




Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Current Model Results

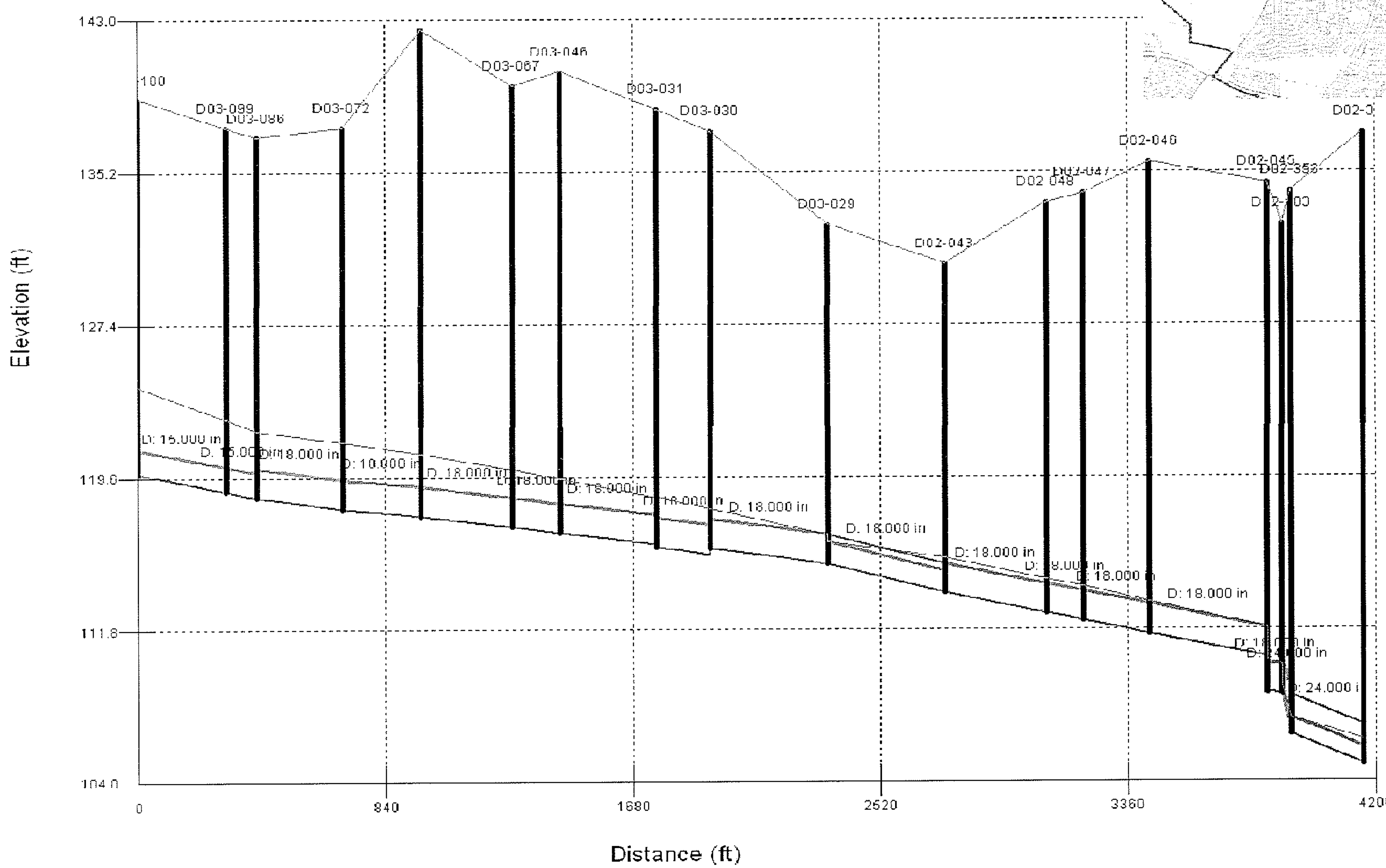
Project Area D



Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Current Model Results

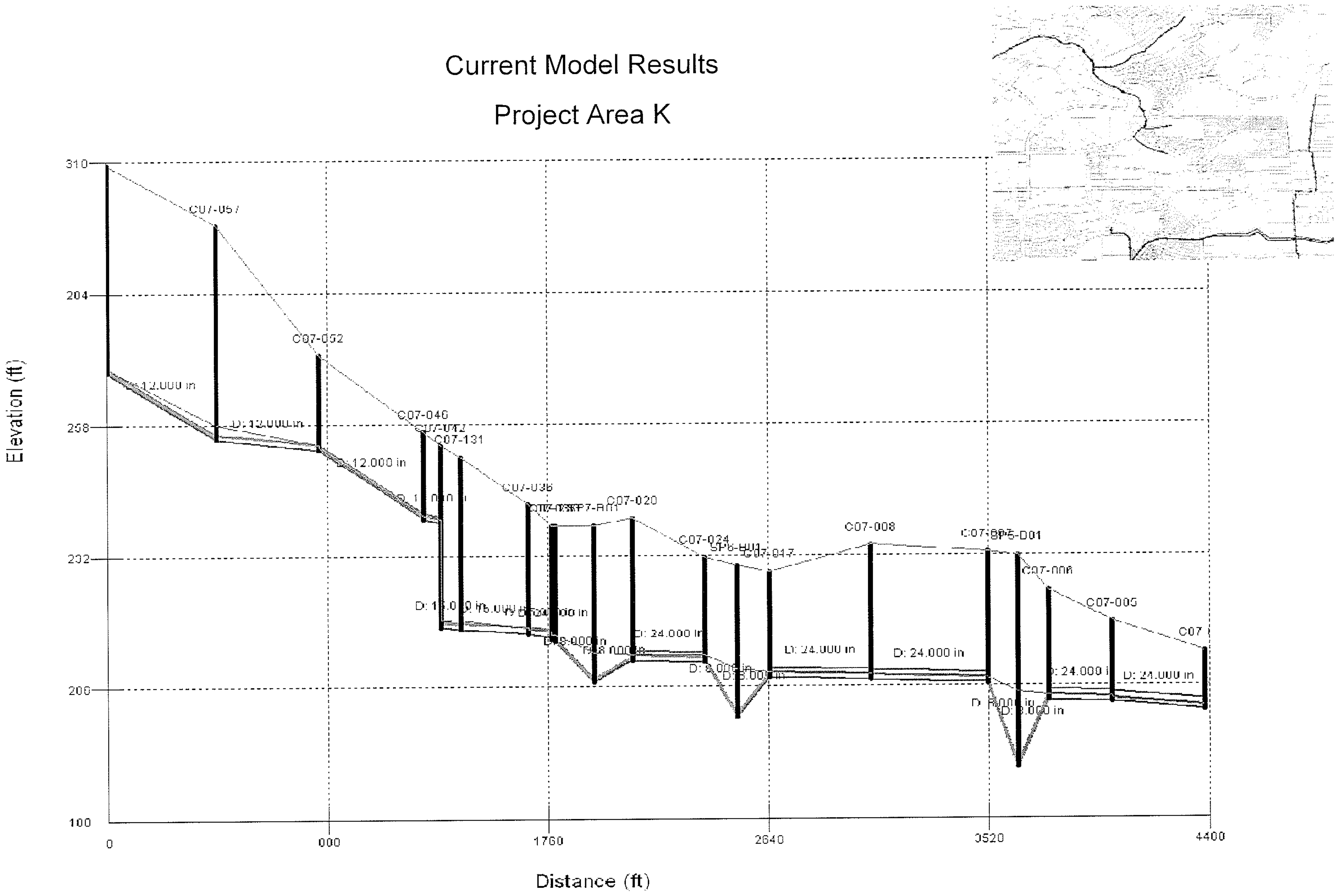
Project Area E



Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Current Model Results

Project Area K



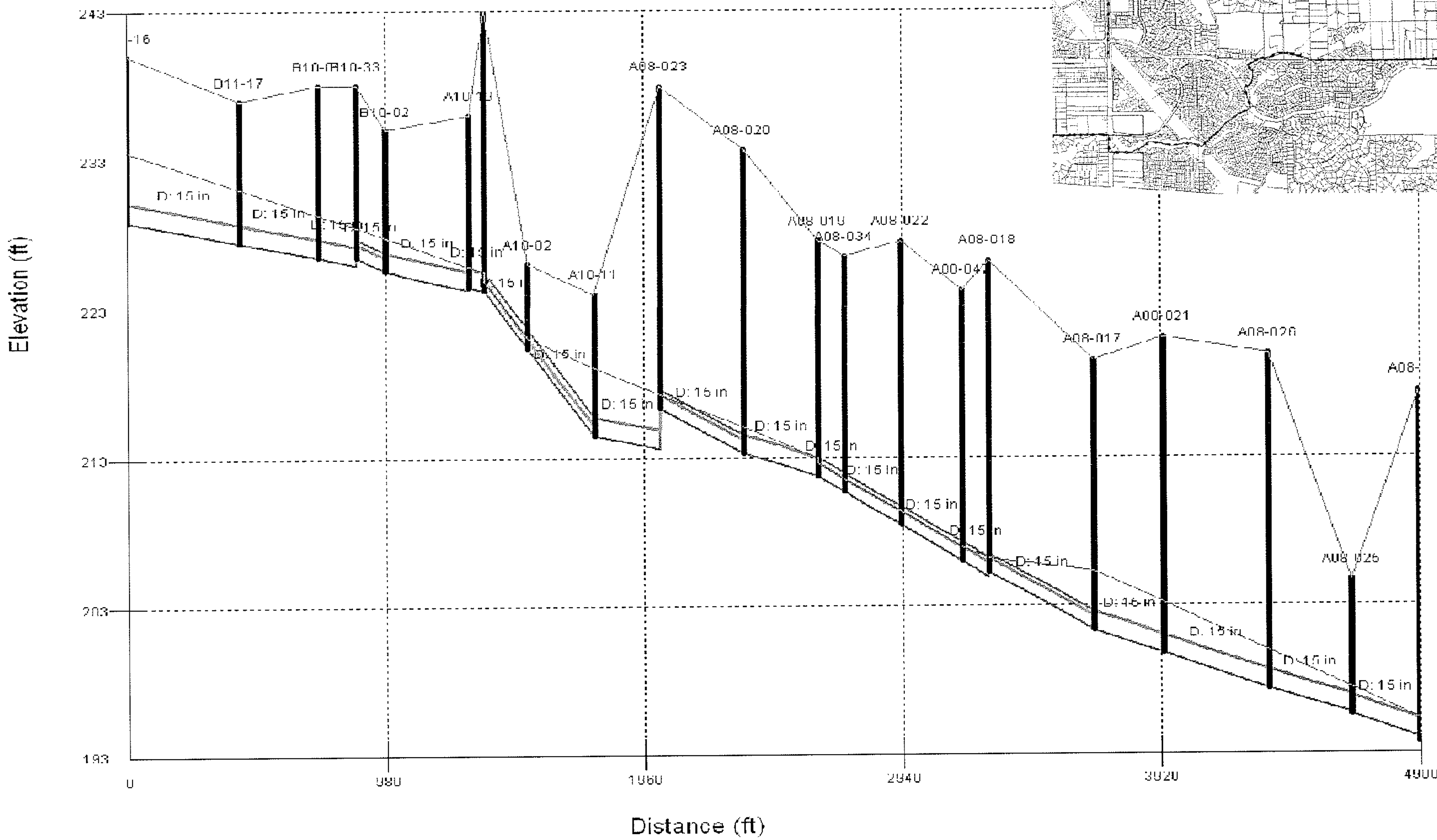
Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

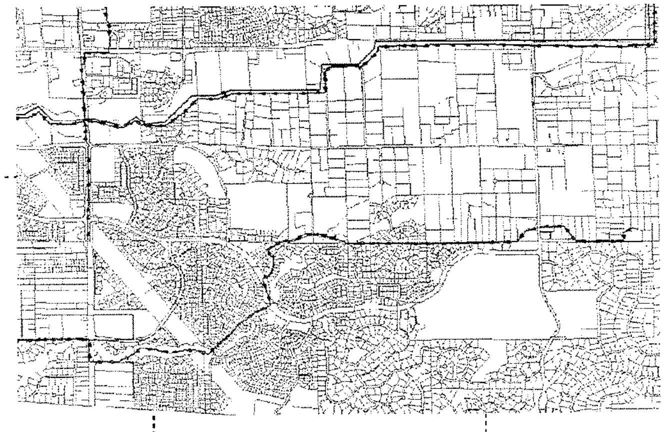
Attachment B

Sewer Profiles – Buildout Scenario

Buildout Model Results

Project Area A

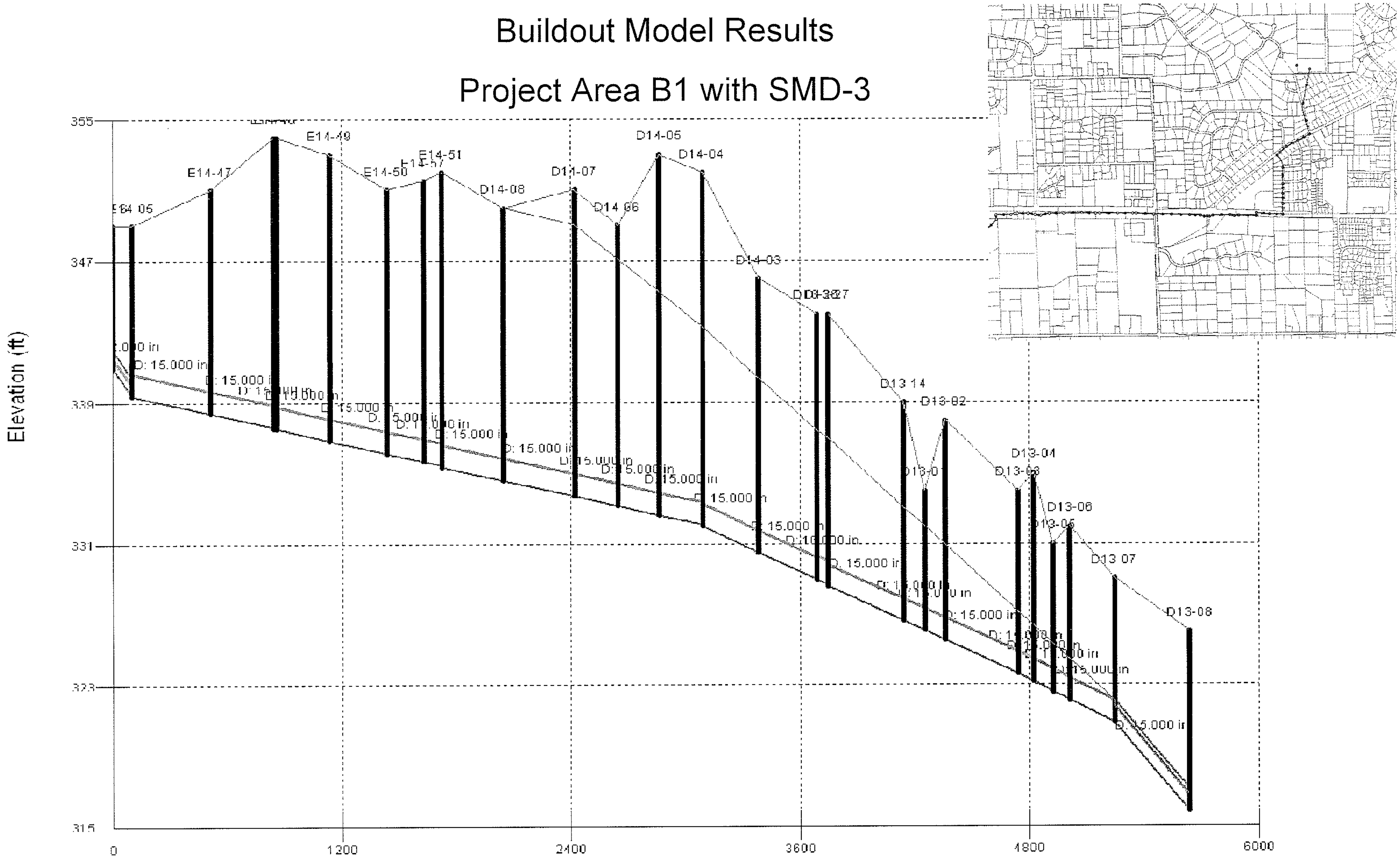




Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

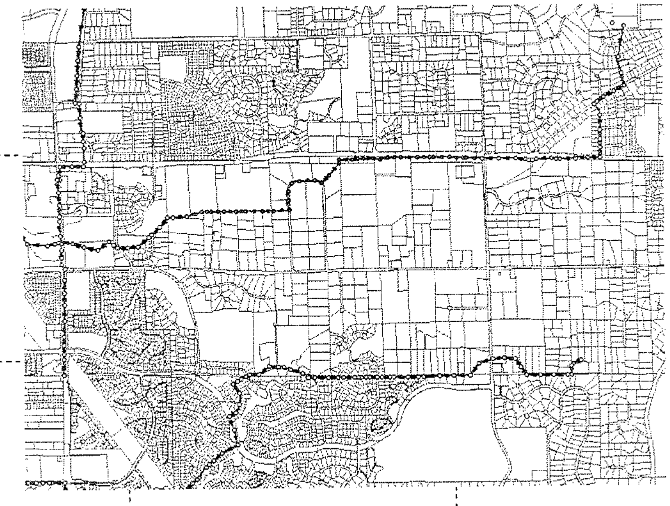
Project Area B1 with SMD-3

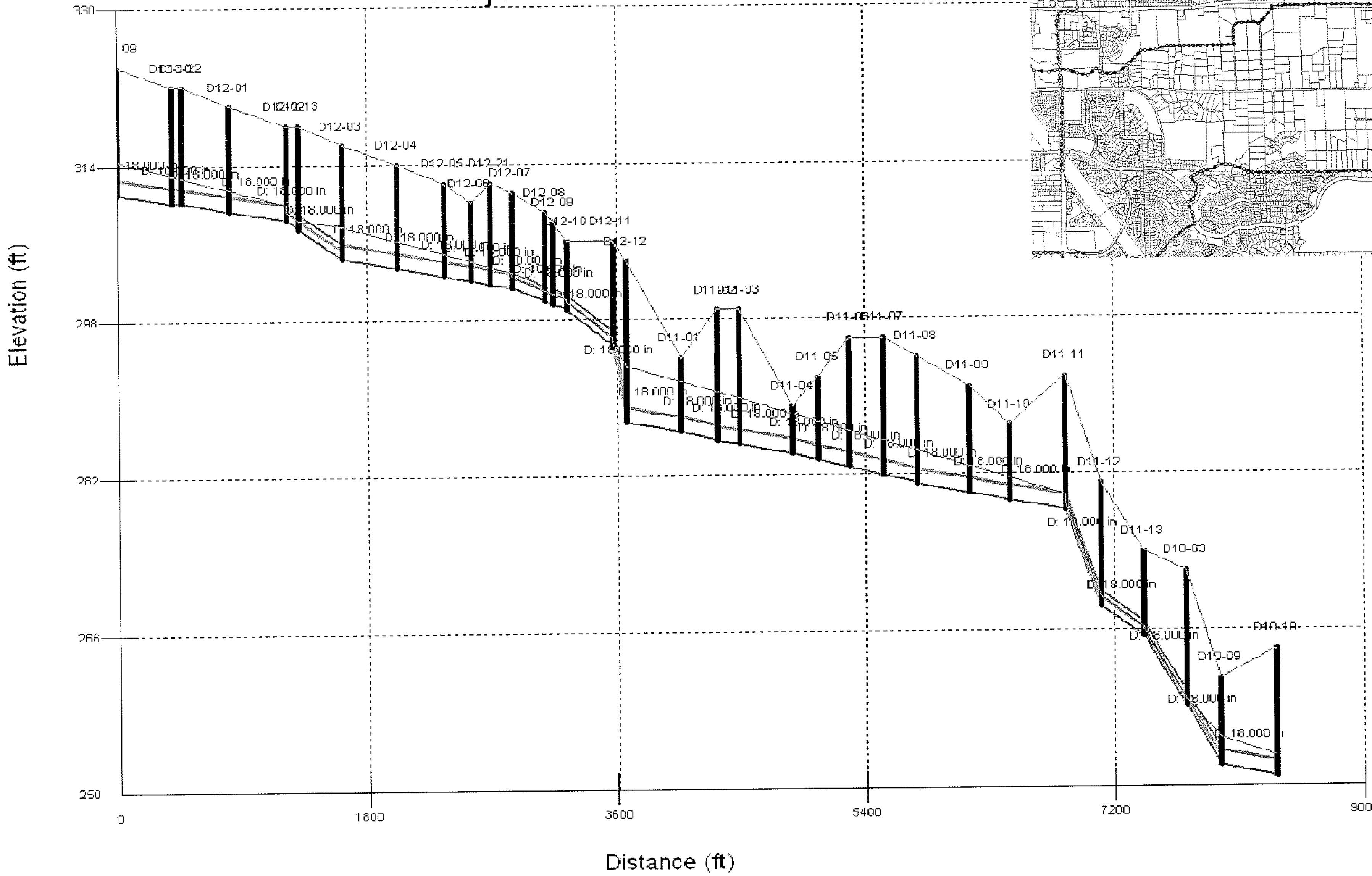


Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

Project Area B1-cont. – with SMD-3

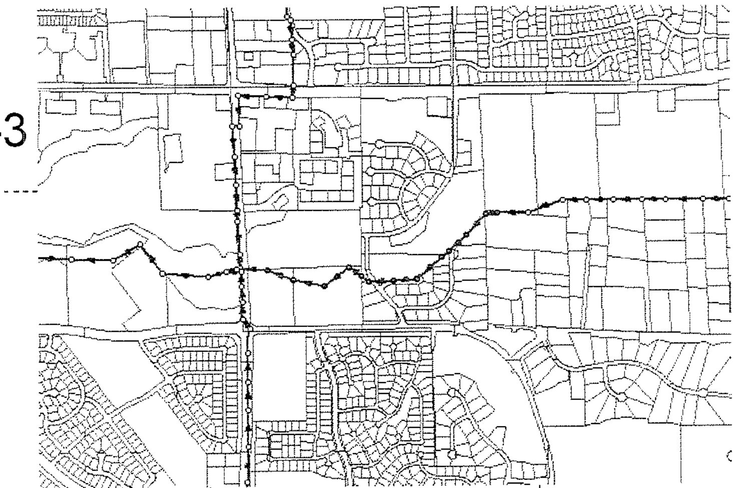


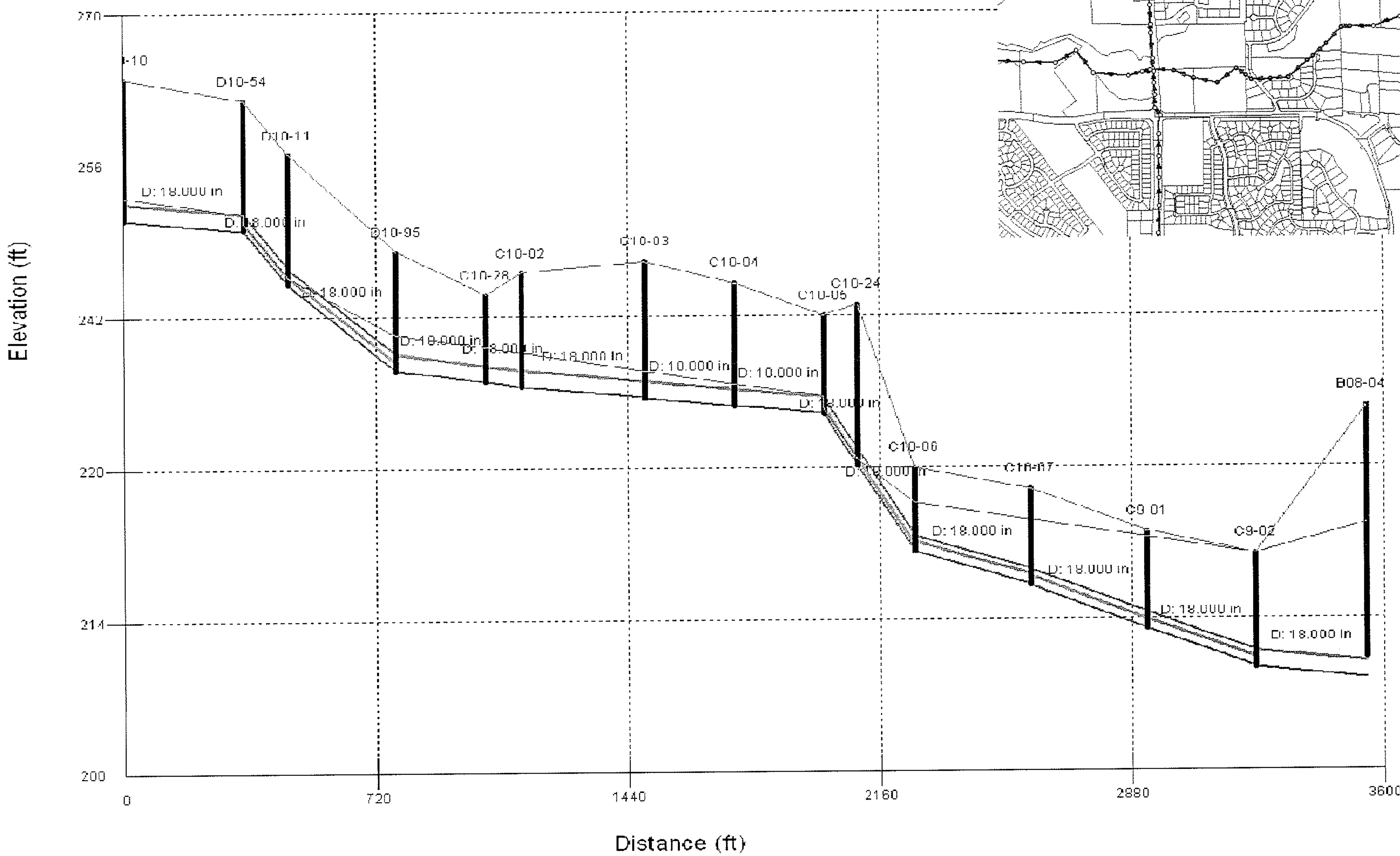


Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

Project Area B1-cont. – with SMD-3

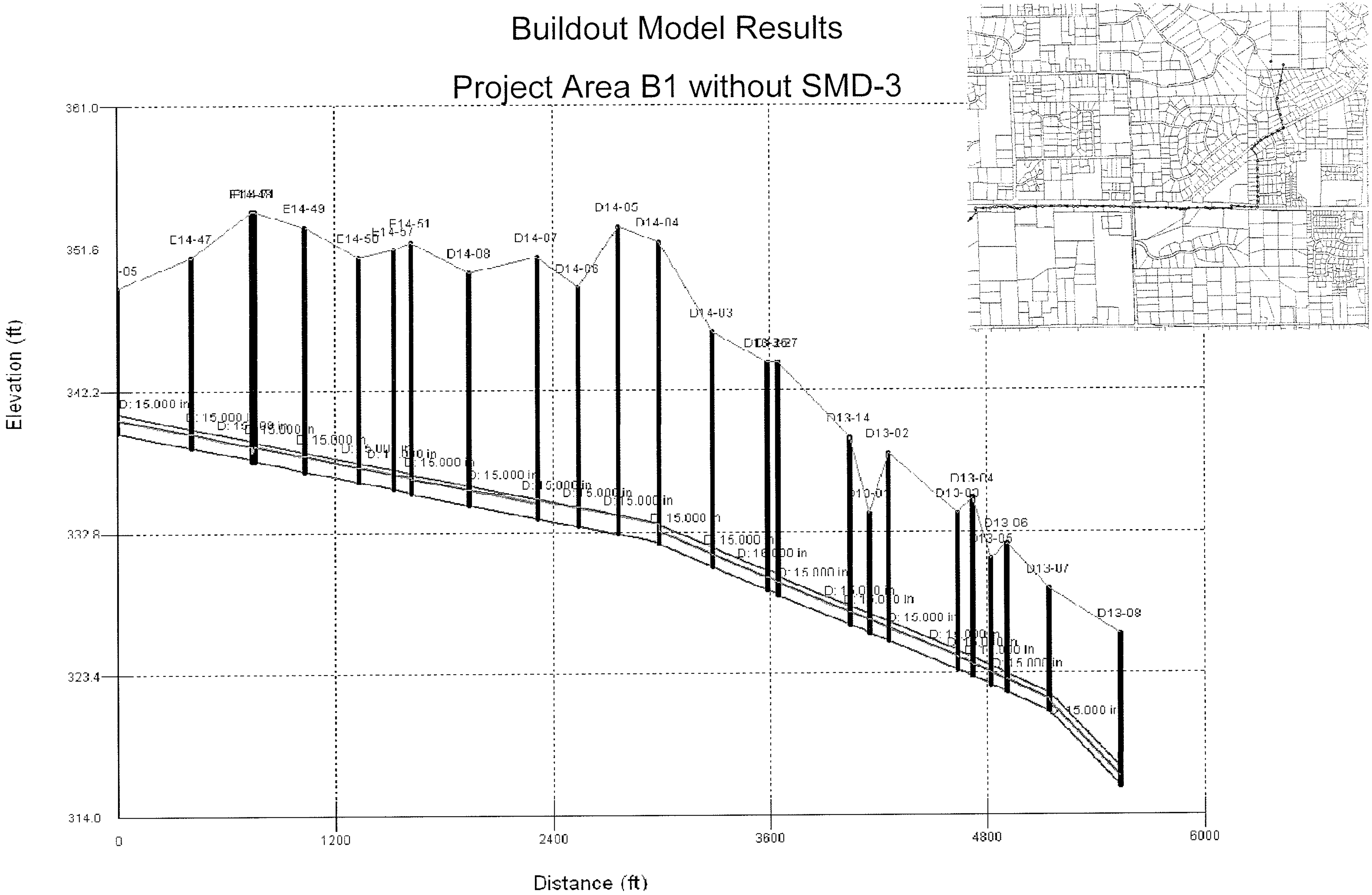




Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

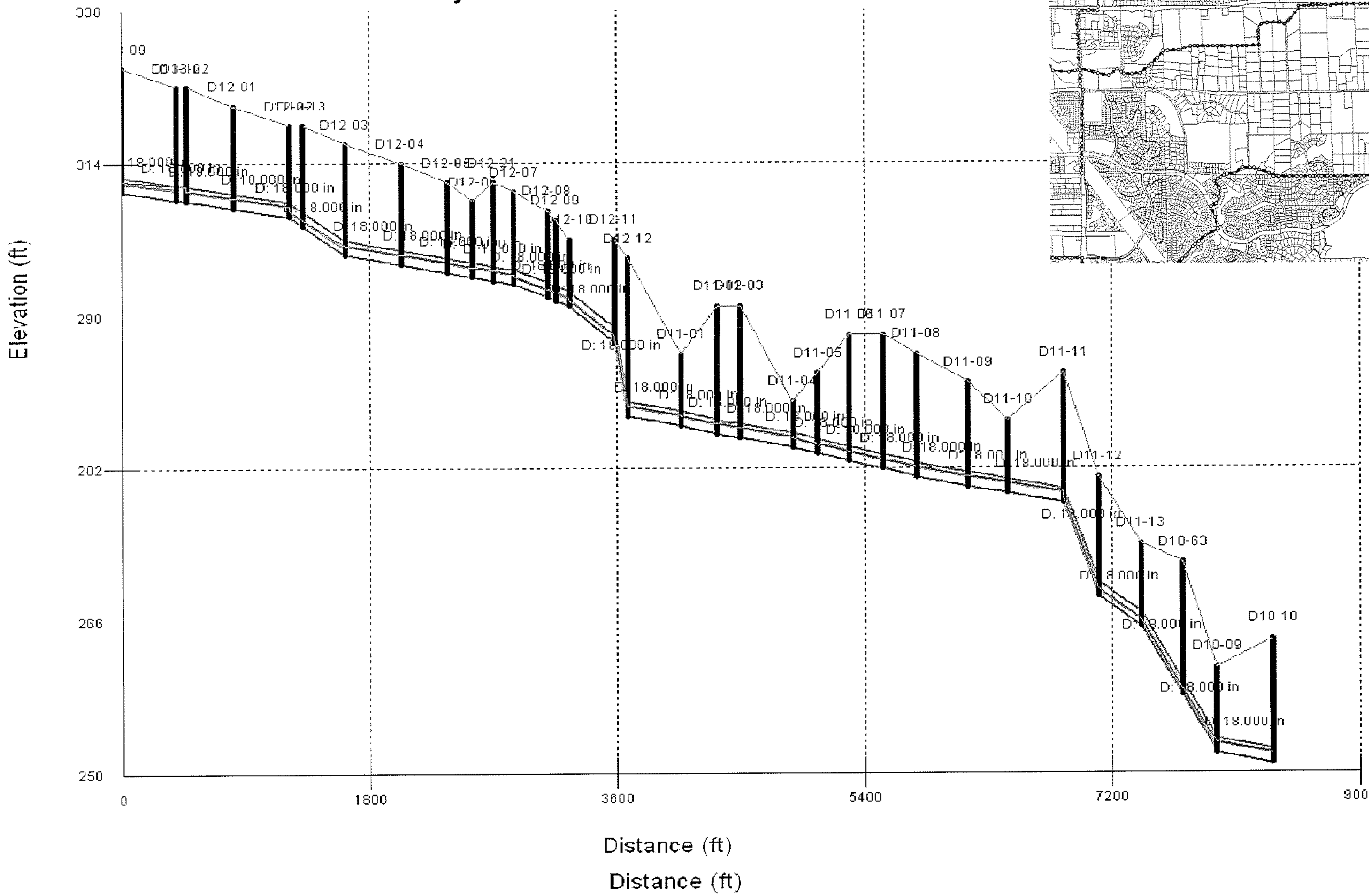
Buildout Model Results

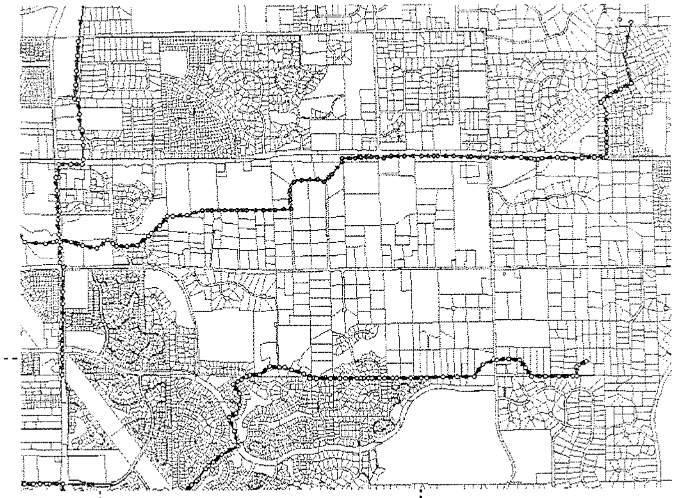
Project Area B1 without SMD-3



Buildout Model Results

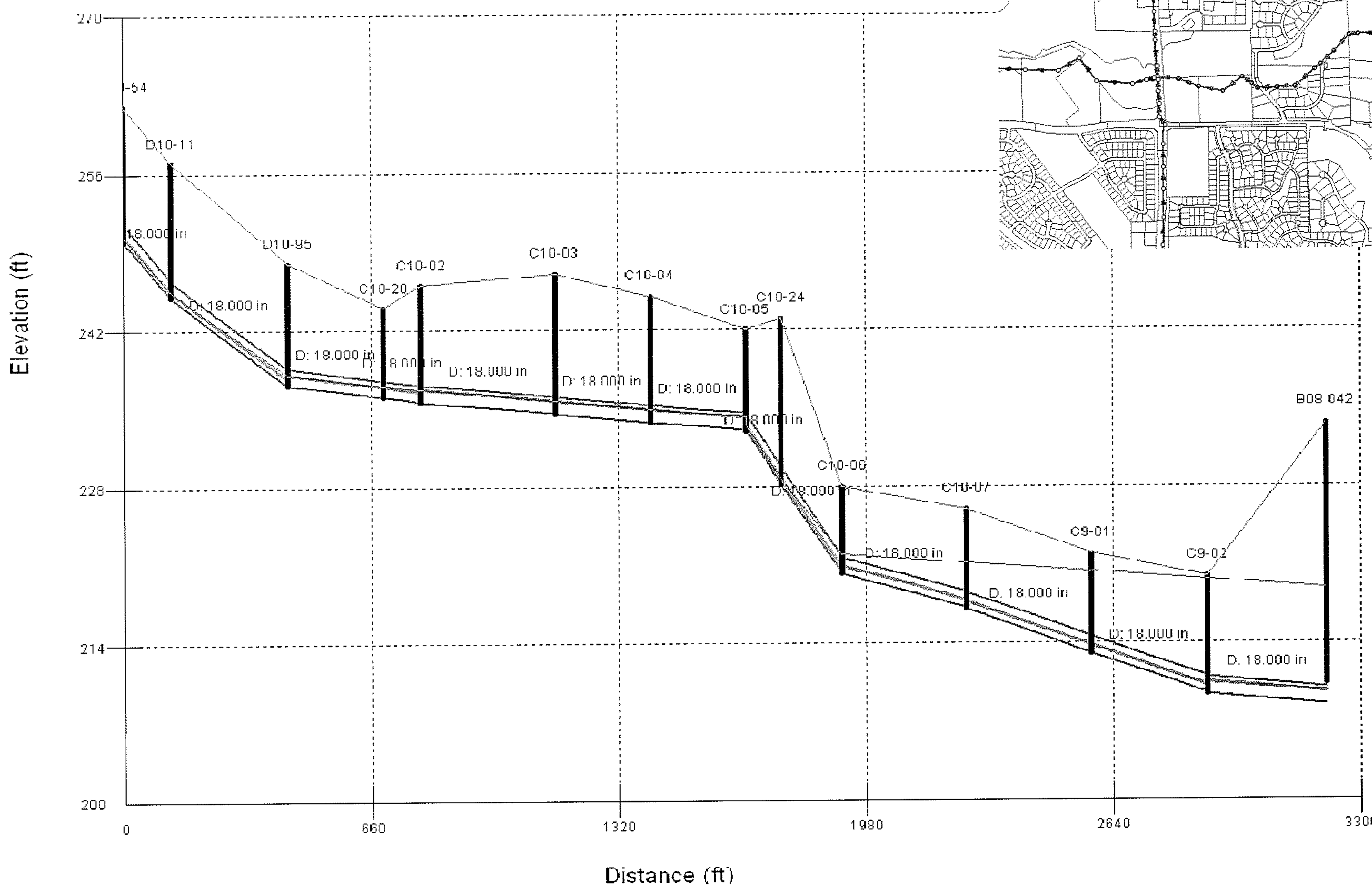
Project Area B1 without SMD-3 -cont.

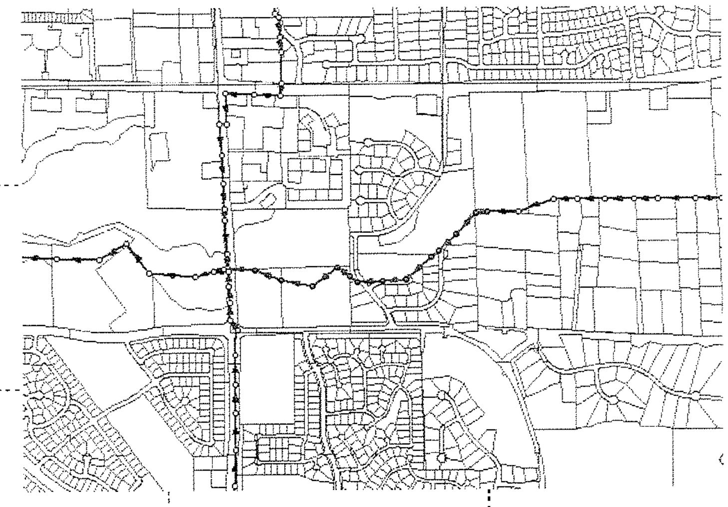




Buildout Model Results

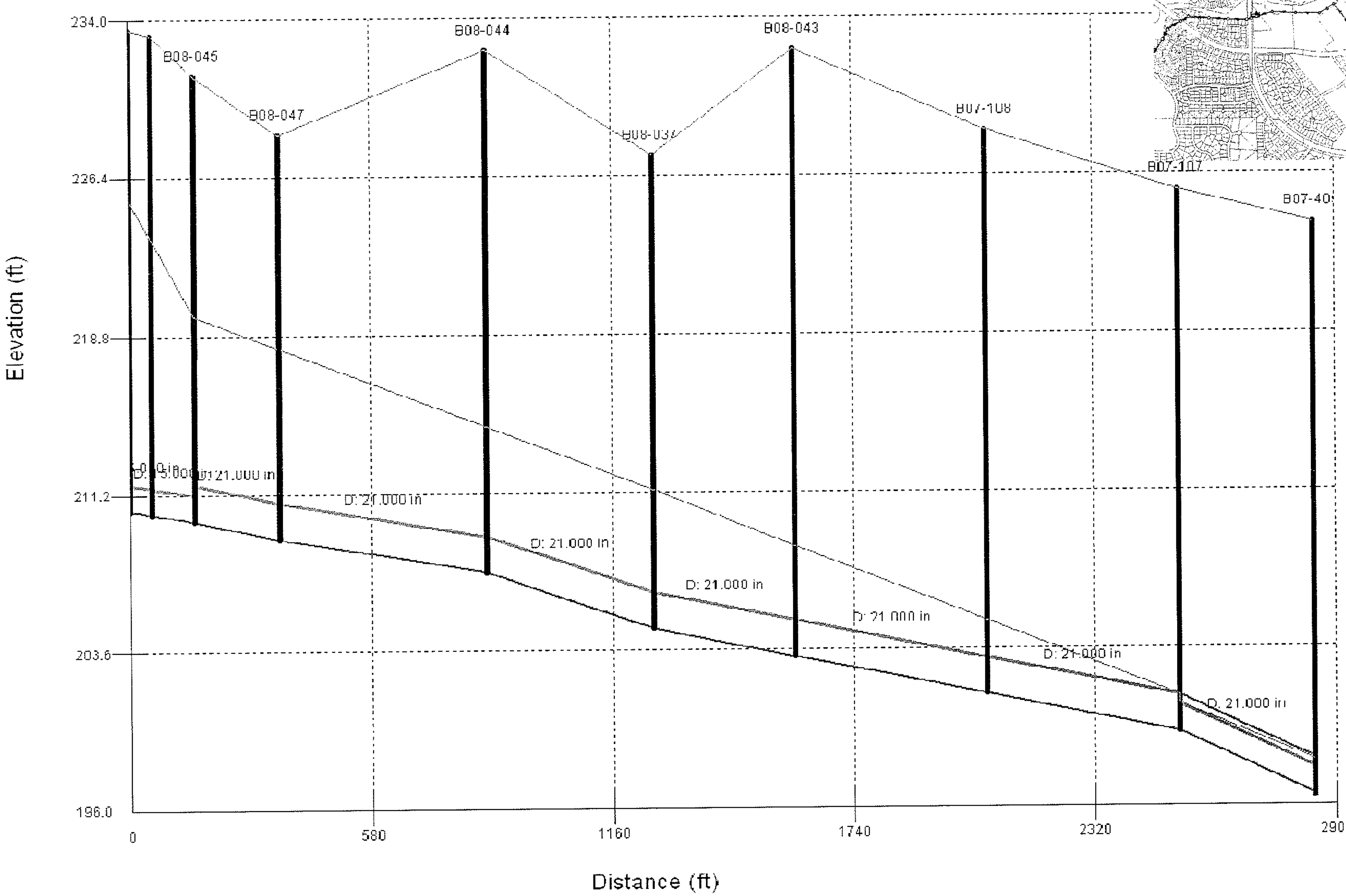
Project Area B1 without SMD-3 –cont.

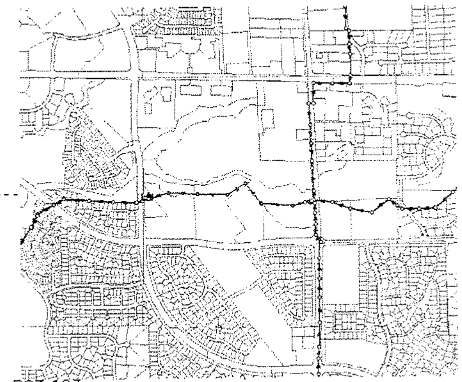




Buildout Model Results

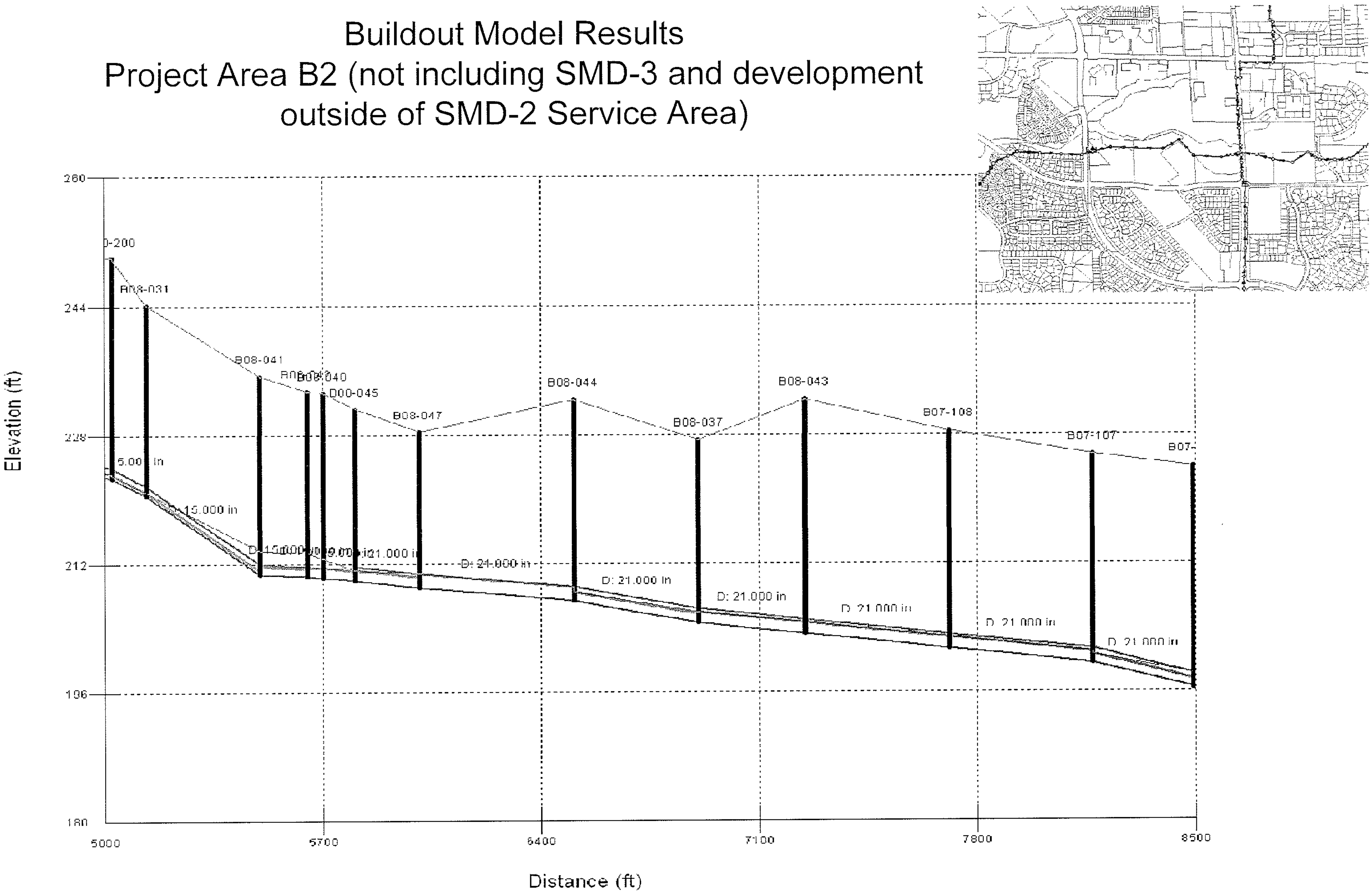
Project Area B2 – with SMD-3





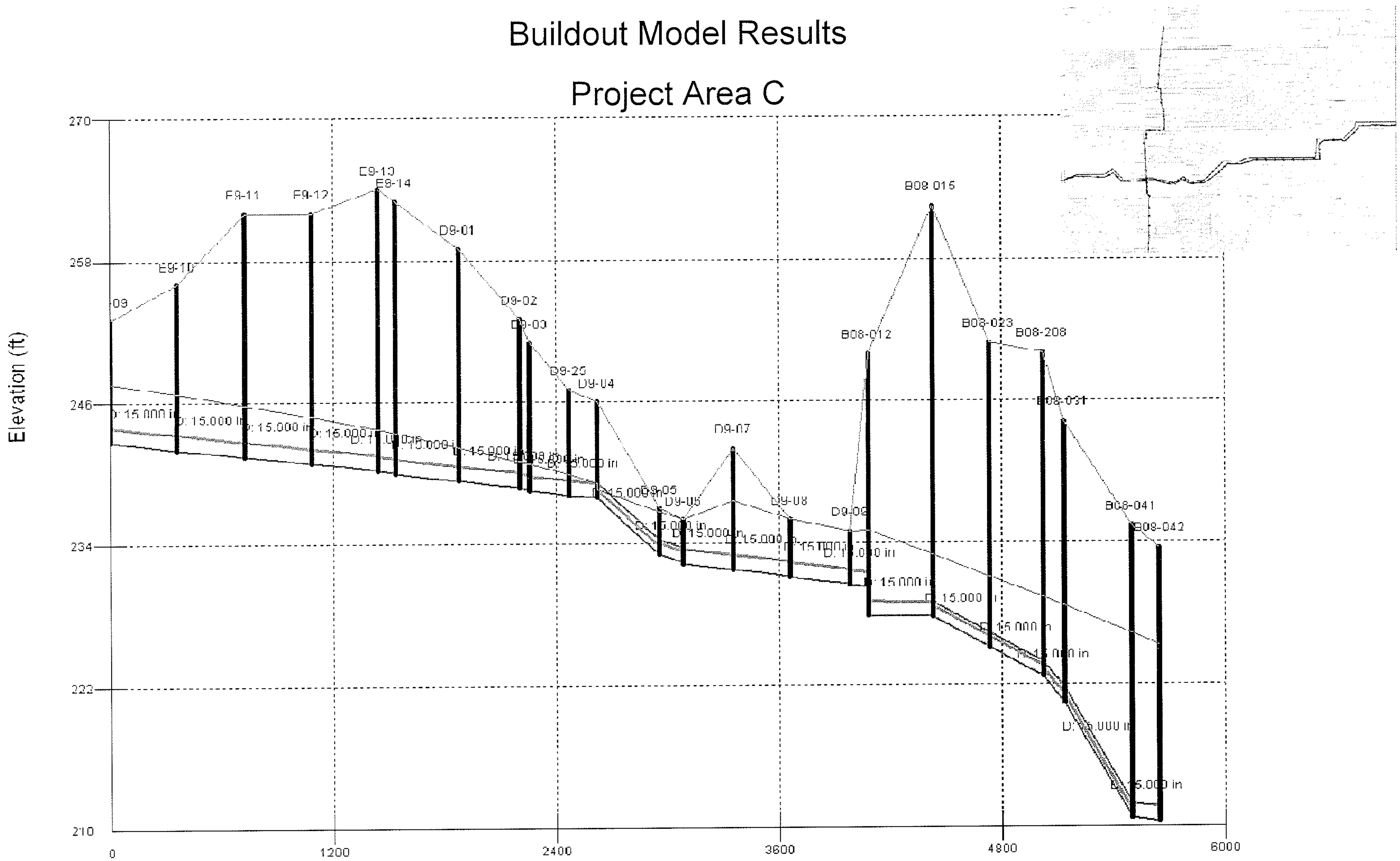
Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results
Project Area B2 (not including SMD-3 and development outside of SMD-2 Service Area)



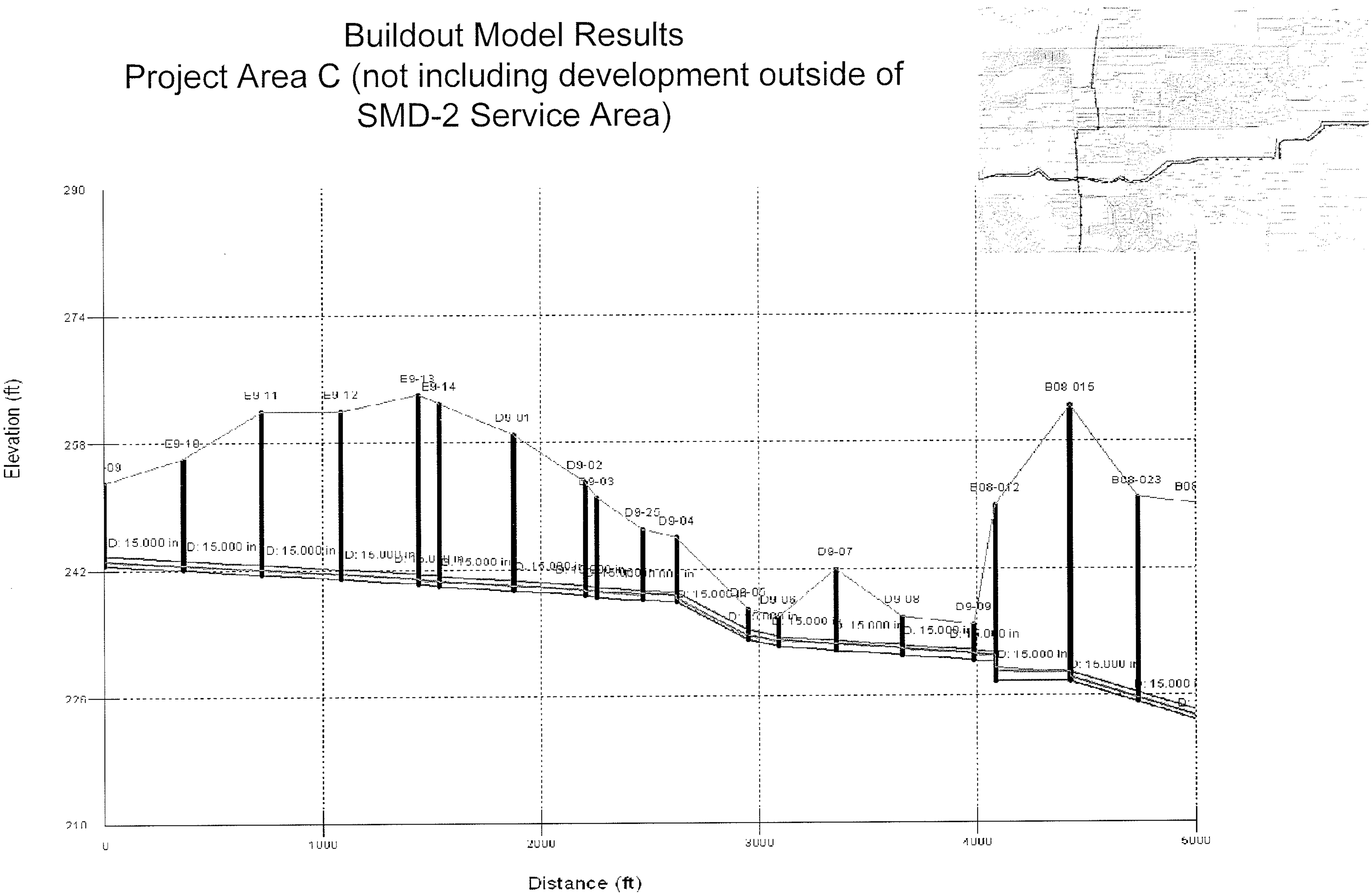
Buildout Model Results

Project Area C



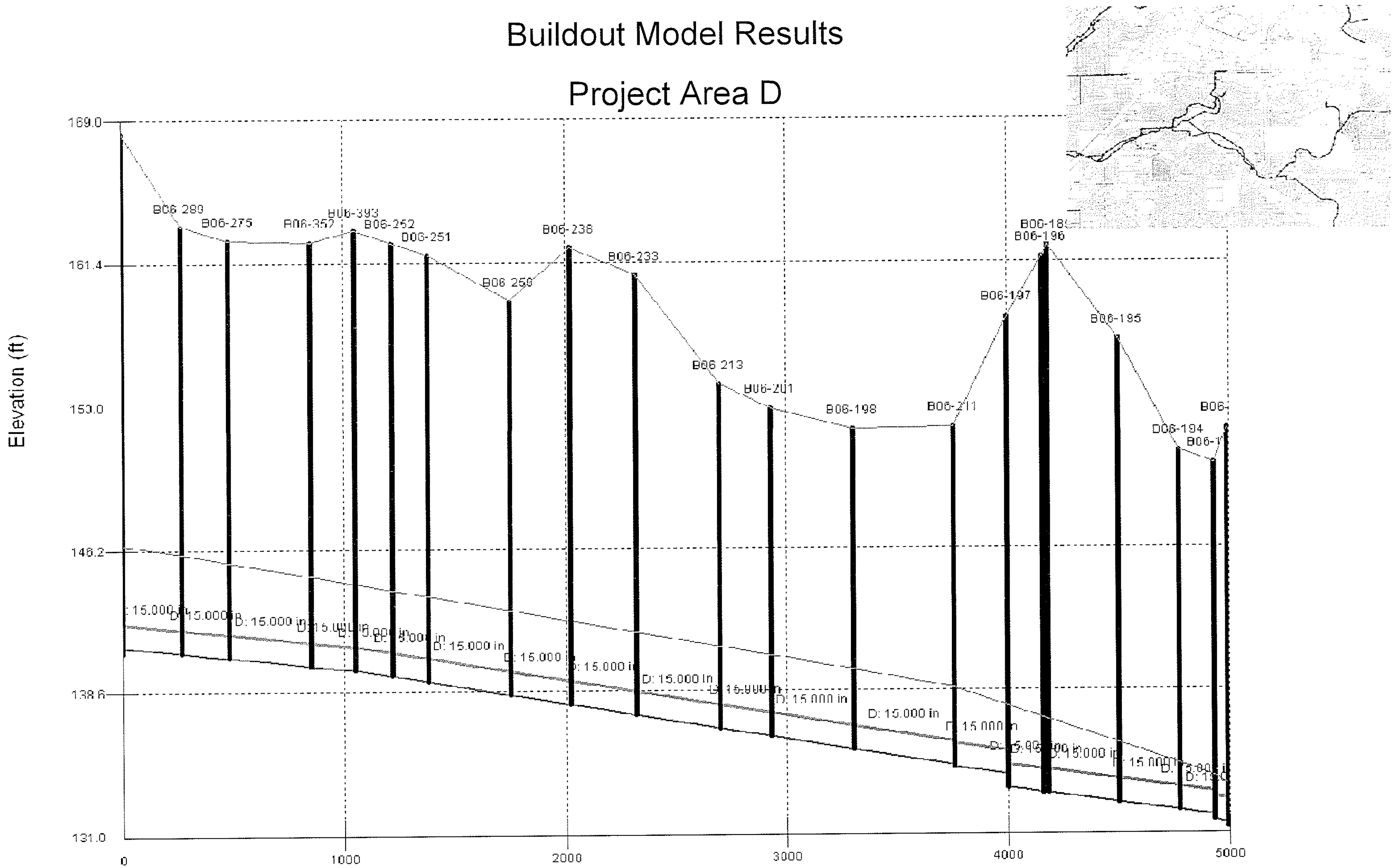
Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results
Project Area C (not including development outside of SMD-2 Service Area)



Buildout Model Results

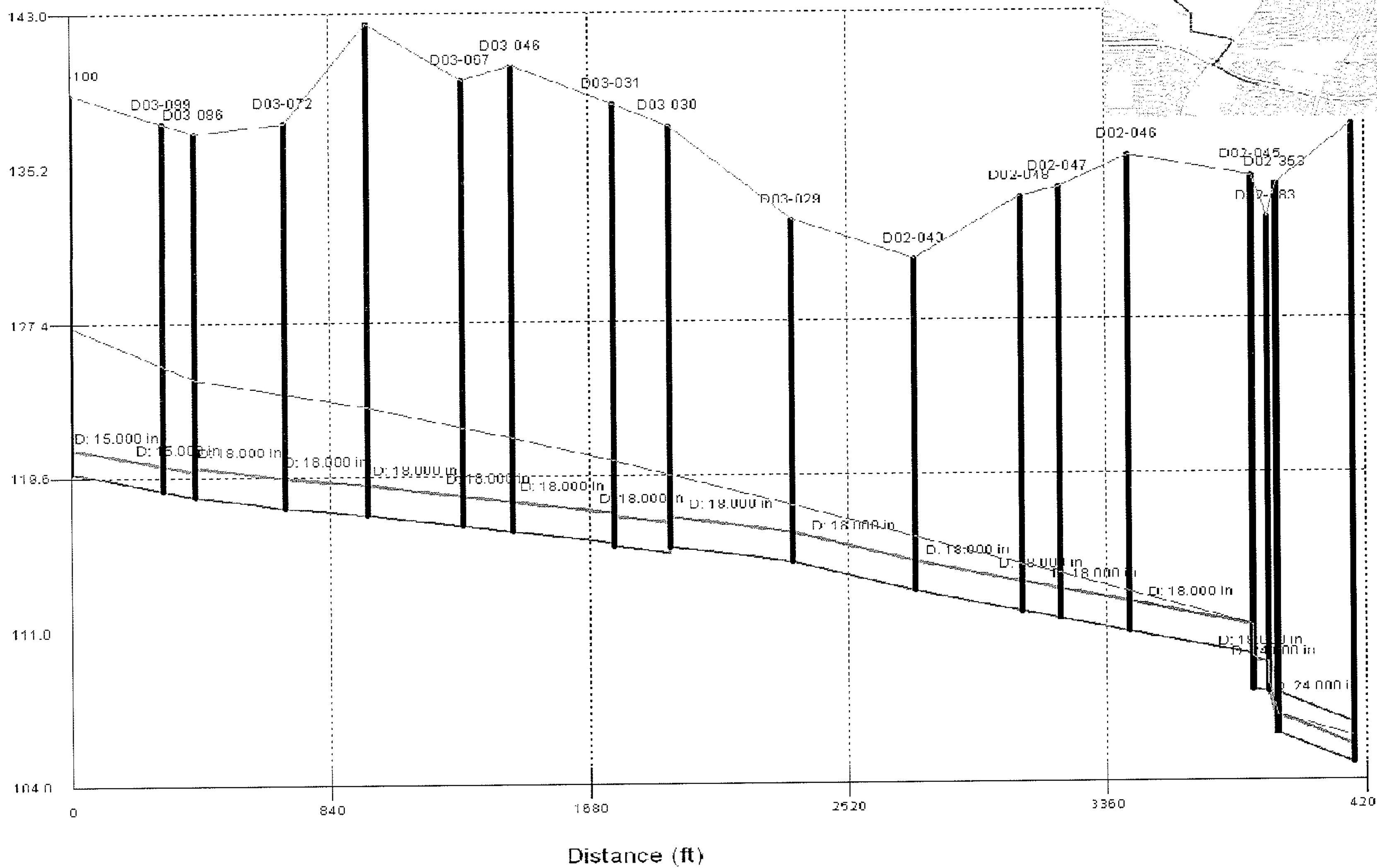
Project Area D



Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

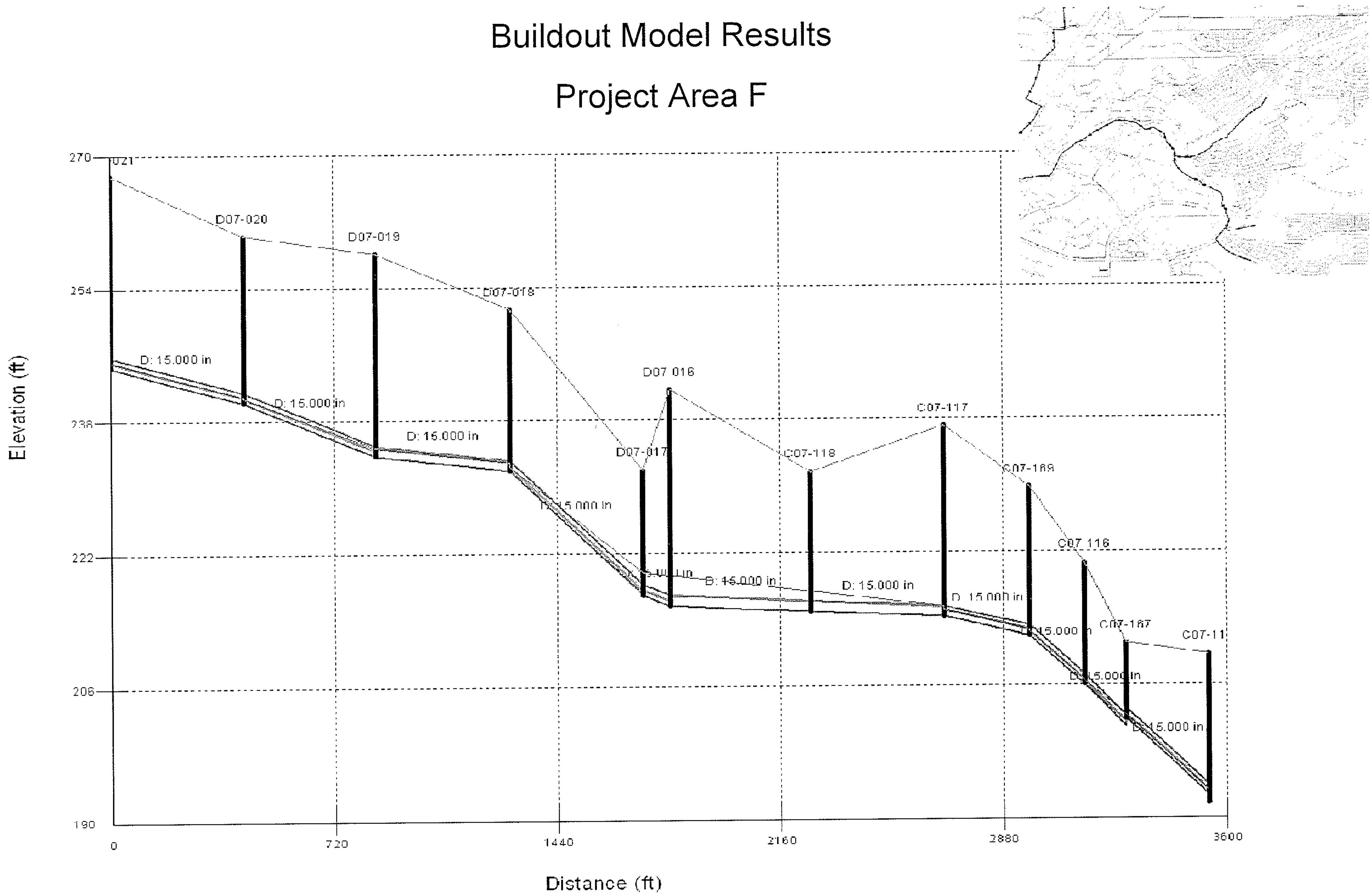
Project Area E



Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

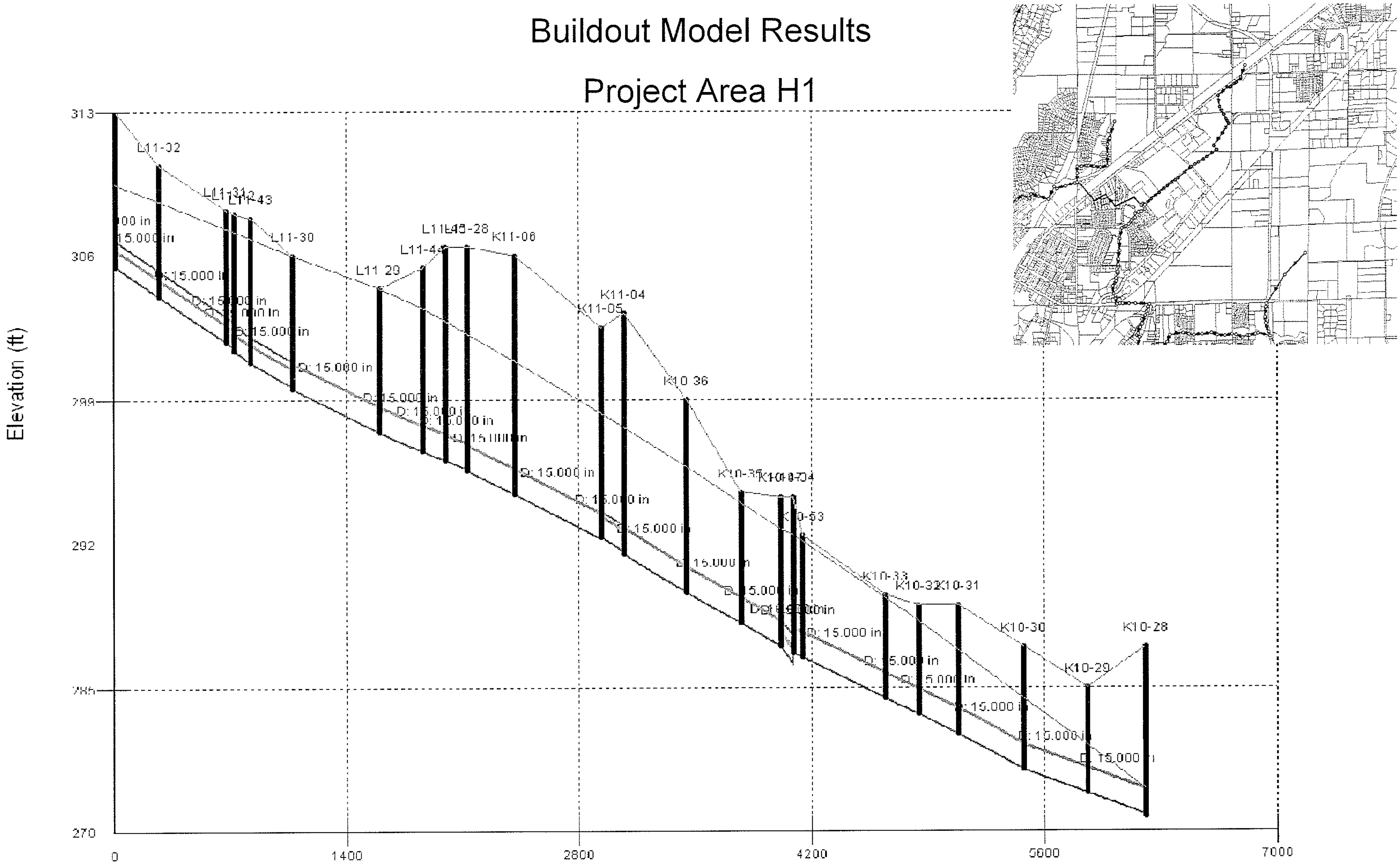
Project Area F



Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

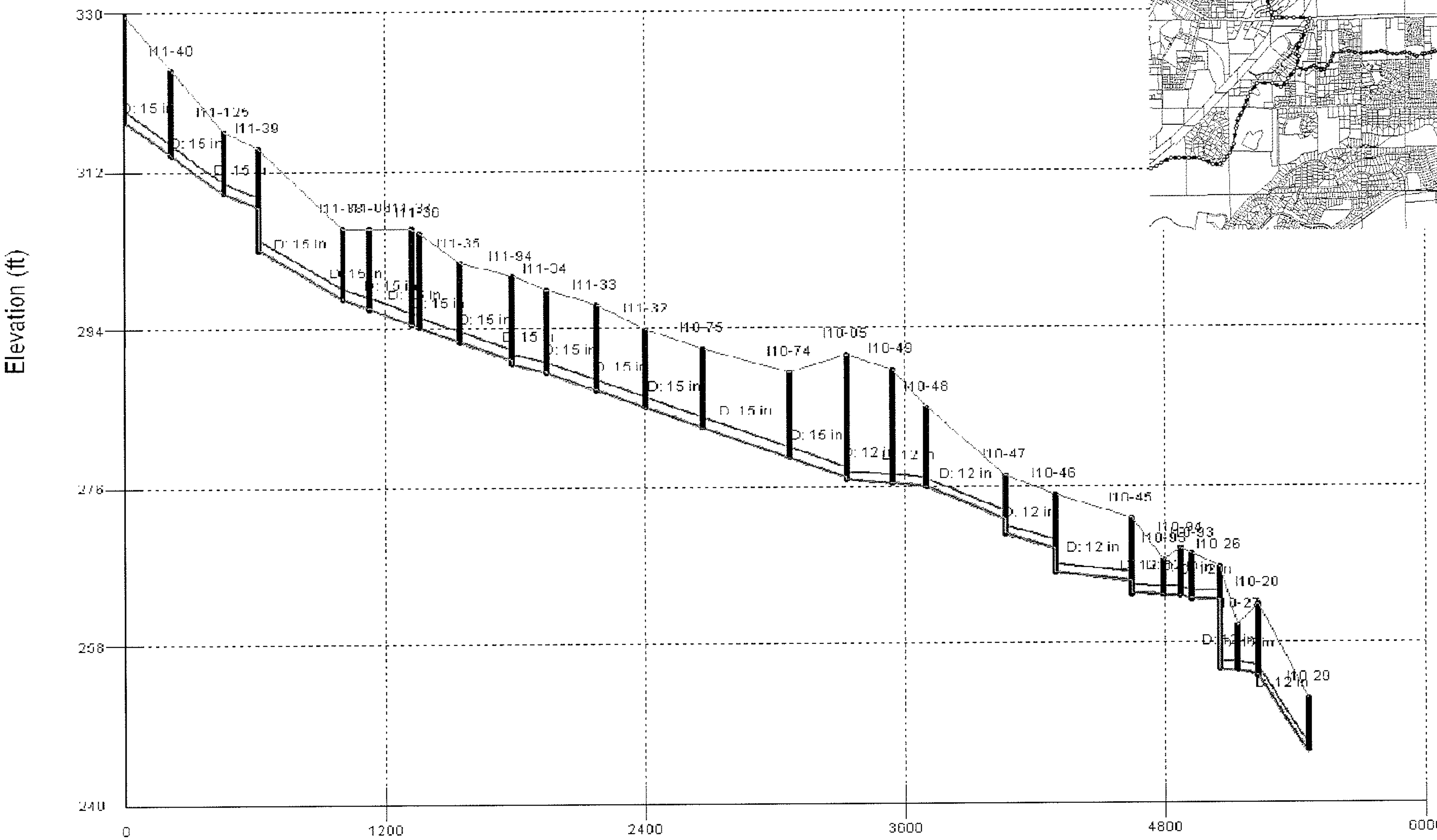
Project Area H1

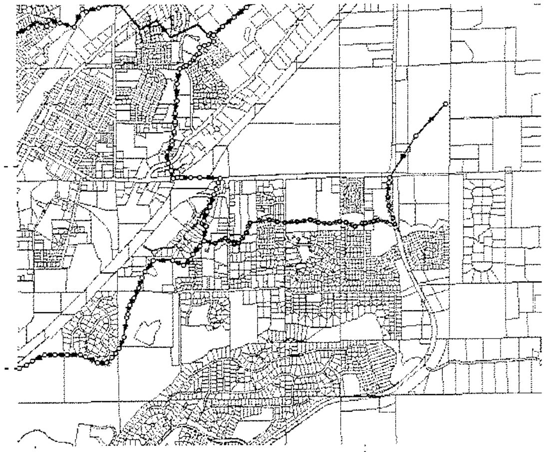


Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

Project Area H2

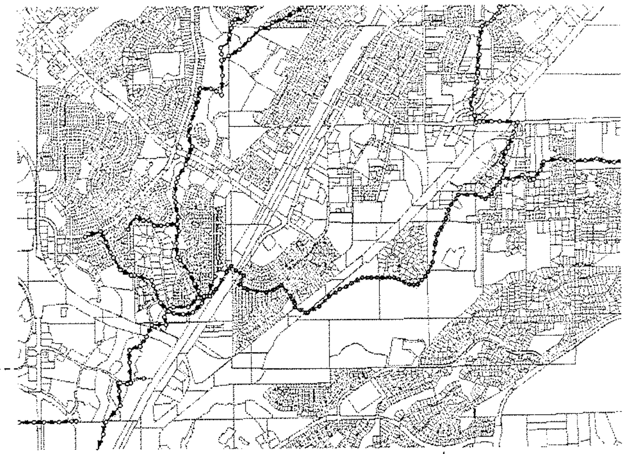


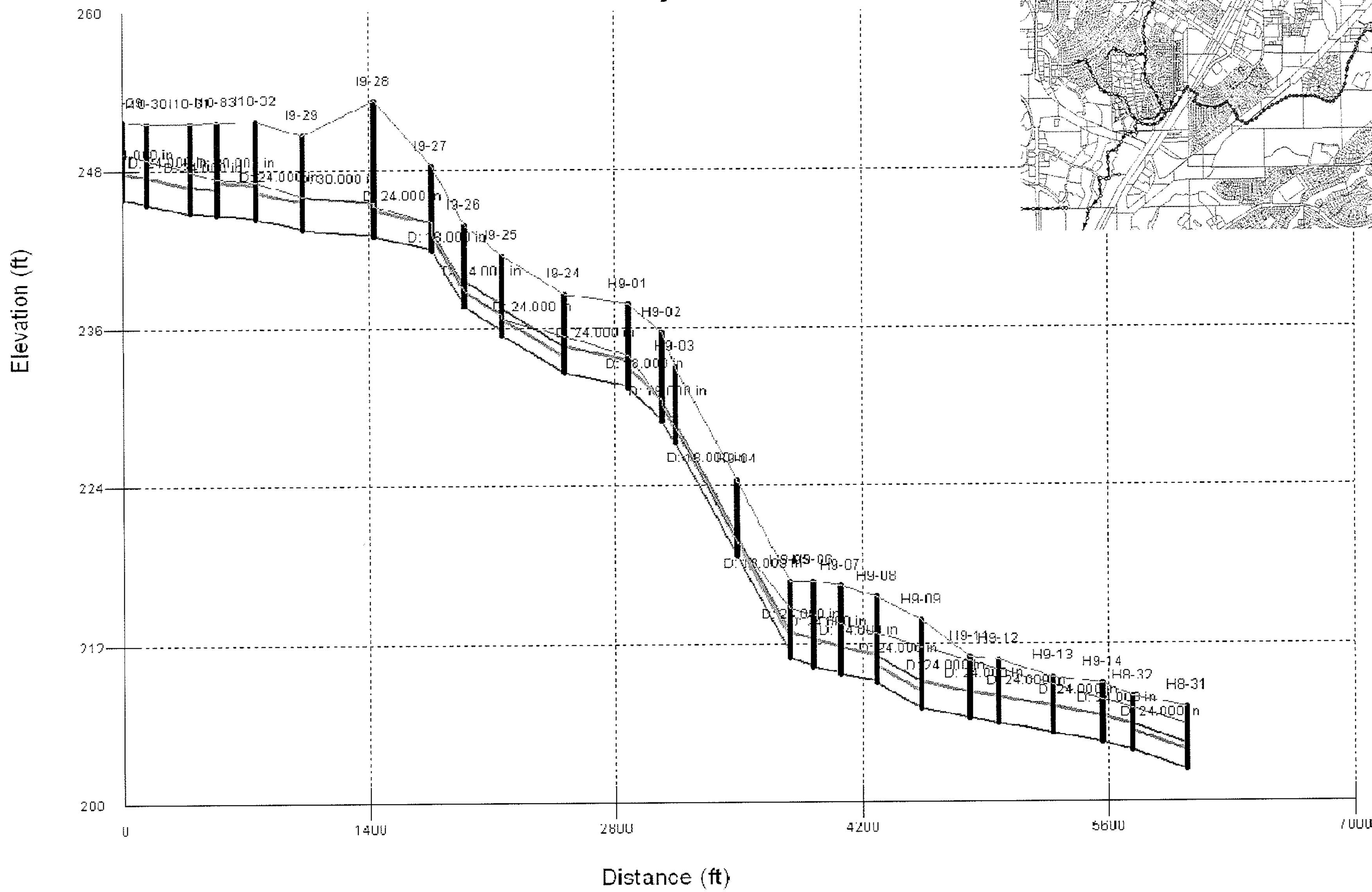


Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

Project Area H3

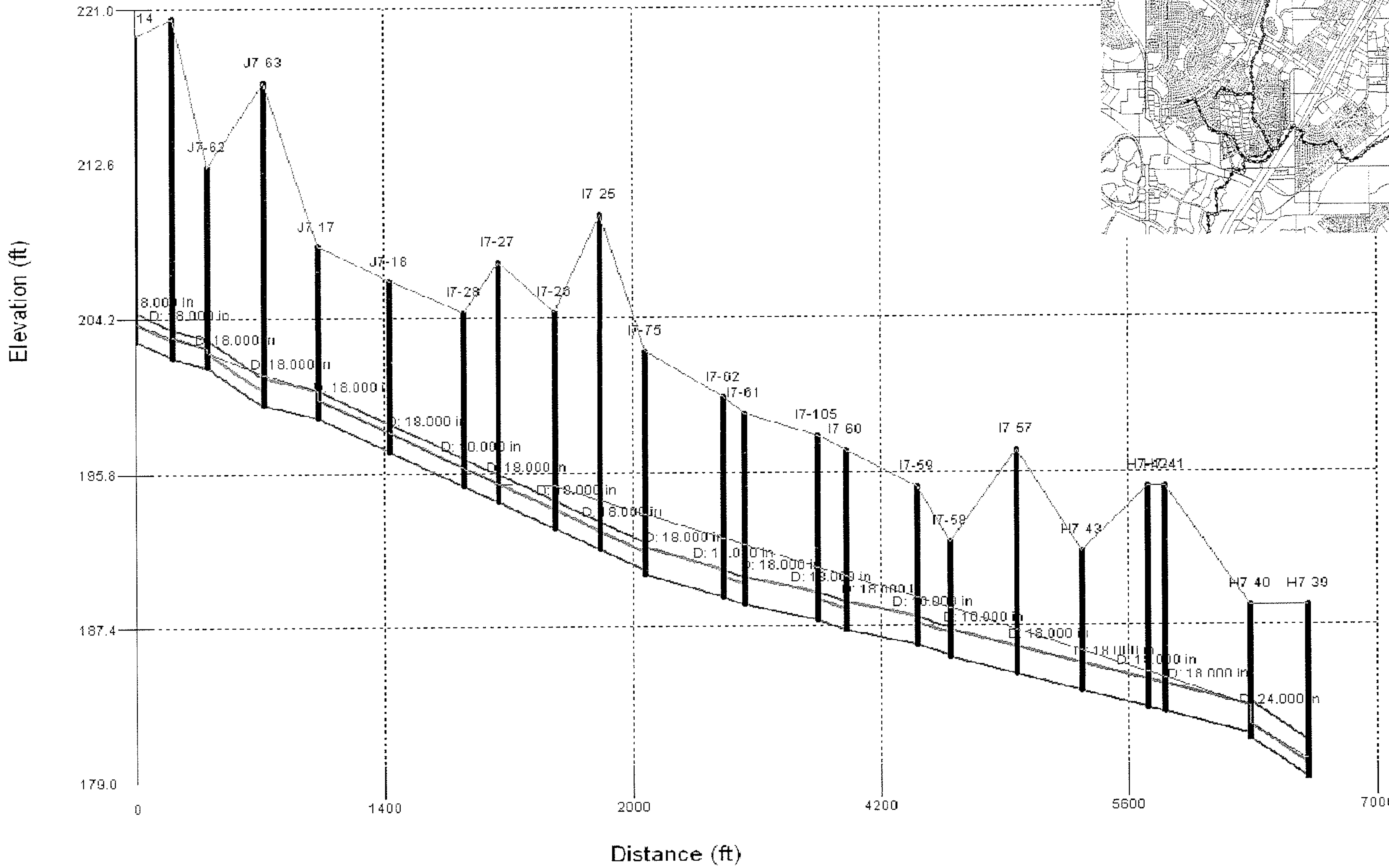


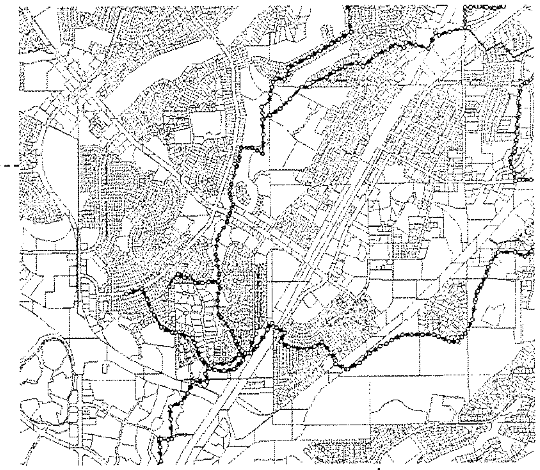


Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

Project Area H4

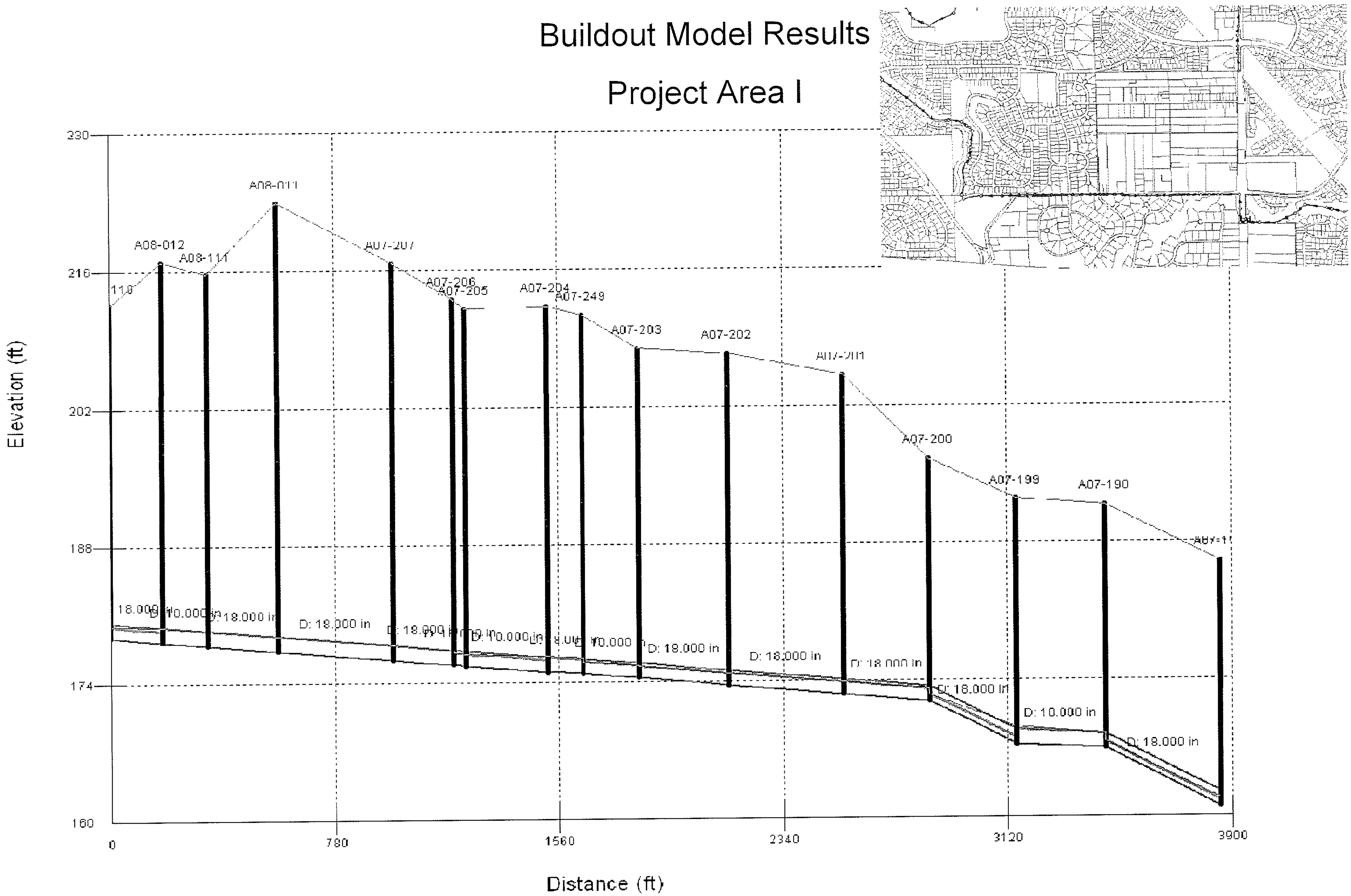




Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

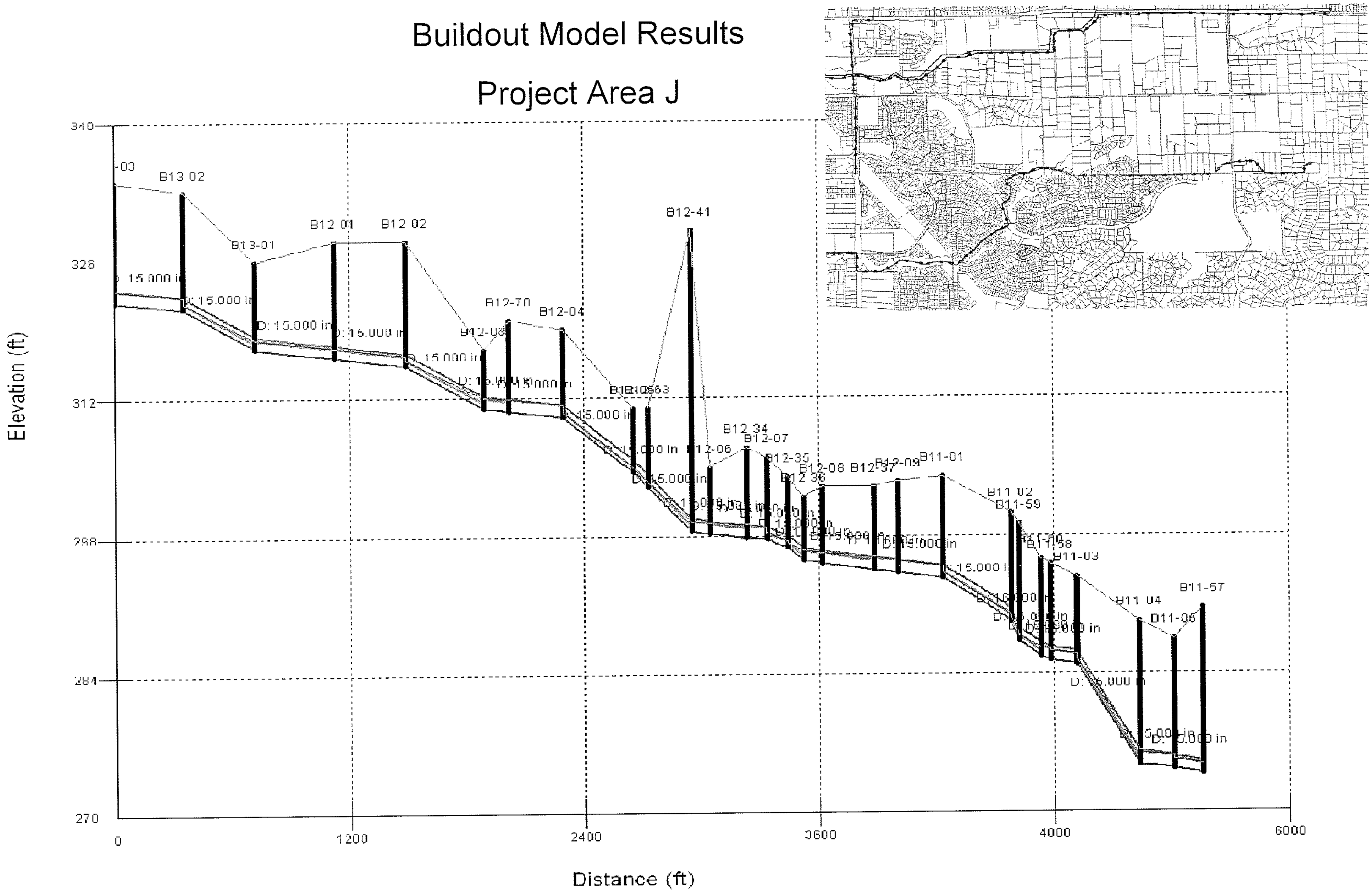
Project Area I



Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

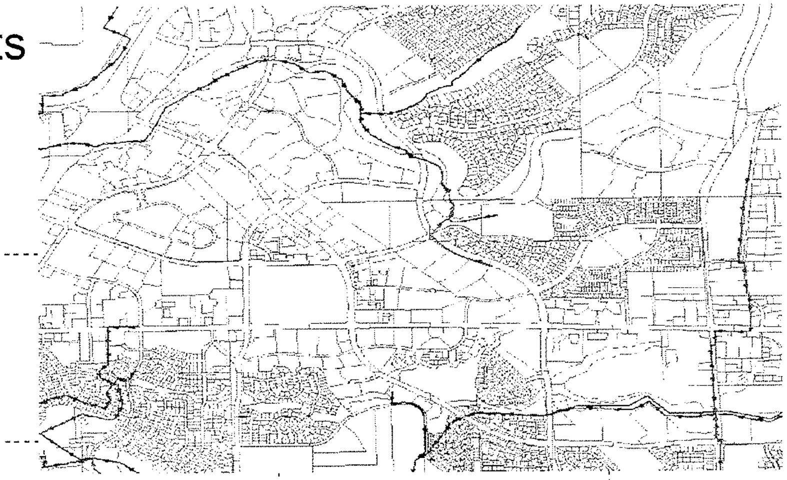
Project Area J

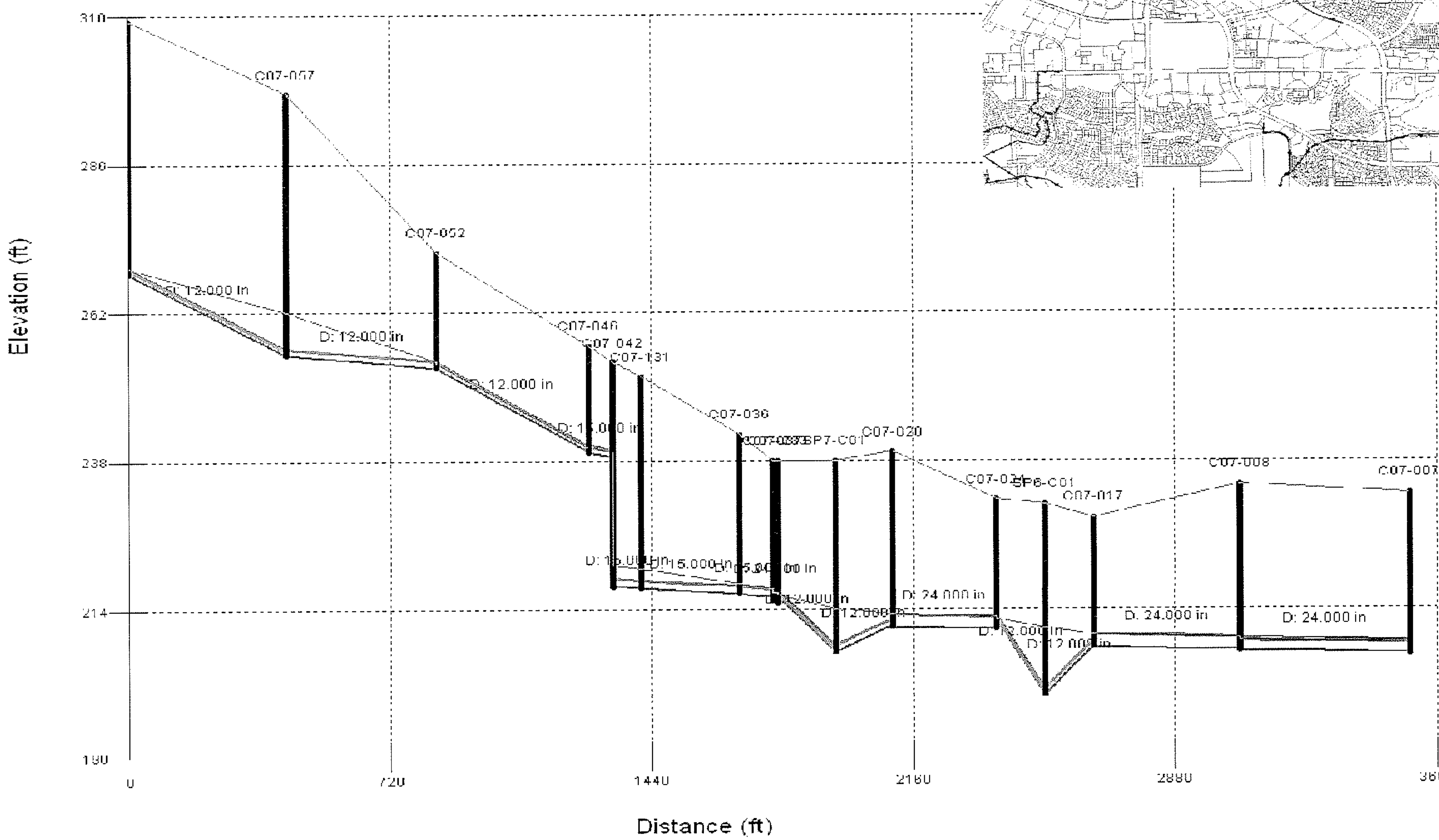


Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Buildout Model Results

Project Area K





Note: Pipe shown in red, HGL shown in purple and ground elevation shown in blue

Attachment C

Existing Facility Project Summary Tables

SPWA Sewer Evaluation

Pipe Capacity Analysis

Project A

Pipe ID	Upstream Manhole	Placer County Upstream Manhole[1]	Downstream Manhole	Placer County Downstream Manhole[1]	Upstream Invert	Downstream Invert	Upstream Manhole Rim Elevation	Upstream Manhole Depth (ft)	Existing Diameter (inches)	Length (feet)	Slope	Existing PWWF (MGD)	Existing PWWF q/Q	Future PWWF (MGD)	Future PWWF q/Q	Manhole HGL Elevation (ft)	Crown Elevation (ft)	Surcharging in Manhole (ft)	Improved Diameter (inches)
B11-16	B11-16	B11-16	B11-17	B11-17	228.92	227.44	240.00	11.08	15	429	0.003	2.63	1.07	3.20	1.30	233.60	230.17	3.43	18
B11-17	B11-17	B11-17	B10-01	B10-01	227.44	226.49	237.00	9.56	15	294	0.003	2.65	1.12	3.22	1.35	231.08	228.69	2.39	18
B10-01	B10-01	B10-01	B10-33	B10-33	226.49	225.93	238.00	11.51	15	146	0.004	2.67	1.03	3.24	1.25	229.33	227.74	1.59	18
B10-33	B10-33	B10-33	B10-02	B10-02	226.49	225.55	238.00	11.51	15	108	0.009	2.69	0.69	3.25	0.83	228.45	227.74	0.71	18
B10-02	B10-02	B10-02	A10-19	A10-19	225.55	224.24	235.00	9.45	15	319	0.004	2.71	1.01	3.28	1.22	227.79	226.80	0.99	18
A10-19	A10-19	A10-19	A10-01	A10-01	224.43	224.23	236.00	11.57	15	56	0.004	2.72	1.09	3.29	1.32	225.83	225.68	0.15	18
A10-01	A10-01	A10-01	A10-02	A10-02	224.24	220.37	243.00	18.76	15	165	0.023	2.72	0.42	3.30	0.51	224.88	225.49	0.00	18
A10-02	A10-02	A10-02	A10-11	A10-11	220.37	214.50	226.00	5.63	15	250	0.023	2.73	0.43	3.31	0.52	221.01	221.62	0.00	18
A10-11	A10-11	A10-11	A08-023	A10-03	214.50	213.62	224.00	9.50	15	250	0.004	2.92	1.18	3.58	1.44	219.07	215.75	0.00	18
11686	A08-023	A10-03	A08-020	A10-04	216.32	213.33	237.90	21.58	15	318	0.009	2.92	0.72	3.58	0.88	217.23	217.57	0.00	18
11688	A08-020	A10-04	A08-019	A10-05	213.33	211.76	233.64	20.31	15	285	0.005	2.92	0.94	3.58	1.15	215.10	214.58	0.00	18
113542	A08-019	A10-05	A08-034	-	211.76	210.70	227.50	15.74	15	101	0.01	2.91	0.68	3.57	0.83	212.63	213.01	0.00	18
113543	A08-034	-	A08-022	A10-06	210.70	208.47	226.48	15.78	15	213	0.011	2.91	0.68	3.57	0.83	211.57	211.95	0.00	18
74279	A08-022	A10-06	A08-047	-	208.47	206.05	227.48	19.02	15	230	0.01	2.91	0.68	3.57	0.83	209.34	209.72	0.00	18
113544	A08-047	-	A08-018	A10-07	206.05	204.96	224.18	18.13	15	104	0.011	3.09	0.72	3.75	0.87	206.95	207.30	0.00	18
11694	A08-018	A10-07	A08-017	A10-08	205.28	201.39	226.16	20.89	15	399	0.01	3.08	0.75	3.75	0.91	206.21	206.53	0.00	18
11696	A08-017	A10-08	A08-021	A10-09	201.39	199.80	219.46	18.08	15	261	0.006	3.13	0.96	3.78	1.16	205.36	202.64	2.72	18
11698	A08-021	A10-09	A08-026	A9-51	199.80	197.37	220.99	21.20	15	398	0.006	3.13	0.96	3.77	1.15	203.22	201.05	2.18	18
11700	A08-026	A9-51	A08-025	A9-01	197.37	195.70	219.85	22.48	15	311	0.005	3.12	1.02	3.77	1.23	199.97	198.62	1.35	18
11702	A08-025	A9-01	A08-156	-	195.60	194.03	204.70	9.10	15	255	0.006	3.11	0.95	3.82	1.16	197.44	196.85	0.59	18
									Total Length	**4,893**									

1. Placer County Manhole IDs are based on County GIS. Corresponding Placer County manhole IDs are estimated and field verification may be required.

SPWA Sewer Evaluation

Pipe Capacity Analysis

Project B1

Pipe ID	Upstream Manhole	Downstream Manhole	Upstream Invert	Downstream Invert	Upstream Manhole Rim Elevation	Upstream Manhole Depth (ft)	Existing Diameter (inches)	Length (feet)	Slope	Existing PWWF (MGD)	Existing PWWF q/Q	Future PWWF (MGD)	Future PWWF q/Q	Manhole HGL Elevation (ft)	Crown Elevation (ft)	Surcharging in Manhole (ft)	Improved Diameter (inches)
E14-05	E14-05	E14-47	339.40	338.42	349.00	9.60	15	412	0.002	1.17	0.58	3.60	1.76	349.00	340.65	8.35	21
E14-47	E14-47	E14-48	338.42	337.64	351.00	12.58	15	329	0.002	1.19	0.58	3.60	1.77	351.00	339.67	11.33	21
E14-48	E14-48	E14-71	337.54	337.35	354.00	16.46	15	16	0.012	1.38	0.30	3.80	0.83	354.00	338.79	15.21	21
E14-71	E14-71	E14-49	337.52	336.85	354.00	16.48	15	276	0.002	1.38	0.67	3.81	1.85	354.00	338.77	15.23	21
E14-49	E14-49	E14-50	336.85	336.14	353.00	16.15	15	298	0.002	1.38	0.68	3.80	1.86	353.00	338.10	14.90	21
E14-50	E14-50	E14-57	336.14	335.68	351.00	14.86	15	194	0.002	1.39	0.68	3.79	1.86	351.00	337.39	13.61	21
E14-57	E14-57	E14-51	335.68	335.45	351.50	15.82	15	96	0.002	1.39	0.68	3.78	1.85	351.50	336.93	14.57	21
E14-51	E14-51	D14-08	335.35	334.59	352.00	16.65	15	322	0.002	1.44	0.71	3.83	1.88	352.00	336.60	15.40	21
D14-08	D14-08	D14-07	334.59	333.70	350.00	15.41	15	374	0.002	1.44	0.71	3.82	1.87	350.00	335.84	14.16	21
D14-07	D14-07	D14-06	333.70	333.17	351.00	17.30	15	225	0.002	1.51	0.74	3.87	1.90	349.04	334.95	14.09	21
D14-06	D14-06	D14-05	333.17	332.65	349.00	15.83	15	220	0.002	1.51	0.74	3.86	1.90	347.11	334.42	12.69	21
D14-05	D14-05	D14-04	332.65	332.11	353.00	20.35	15	227	0.002	1.54	0.76	3.88	1.90	345.23	333.90	11.33	21
D14-04	D14-04	D14-03	332.01	330.50	352.00	19.99	15	296	0.005	1.70	0.57	4.13	1.38	343.26	333.26	10.00	21
D14-03	D14-03	D13-26	330.50	328.96	346.00	15.50	15	302	0.005	1.70	0.57	4.12	1.38	340.37	331.75	8.62	21
D13-26	D13-26	D13-27	328.96	328.66	344.00	15.04	16	60	0.005	1.71	0.49	4.13	1.17	337.42	330.29	7.13	21
D13-27	D13-27	D13-14	328.56	326.60	344.00	15.44	15	395	0.005	1.72	0.59	4.16	1.41	337.01	329.81	7.20	21
D13-14	D13-14	D13-01	326.60	326.07	339.00	12.40	15	107	0.005	1.73	0.59	4.15	1.41	333.08	327.85	5.23	21
D13-01	D13-01	D13-02	326.07	325.54	334.00	7.93	15	107	0.005	1.73	0.59	4.16	1.41	332.02	327.32	4.70	21
D13-02	D13-02	D13-03	325.54	323.62	338.00	12.46	15	384	0.005	1.74	0.59	4.17	1.41	330.96	326.79	4.17	21
D13-03	D13-03	D13-04	323.62	323.17	334.00	10.38	15	81	0.006	1.74	0.56	4.16	1.34	327.13	324.87	2.26	21
D13-04	D13-04	D13-05	323.17	322.60	335.00	11.83	15	102	0.006	1.75	0.56	4.18	1.34	326.32	324.42	1.90	21
D13-05	D13-05	D13-06	322.60	322.12	331.00	8.40	15	86	0.006	1.75	0.56	4.19	1.34	325.30	323.85	1.44	21
D13-06	D13-06	D13-07	322.12	320.83	332.00	9.88	15	232	0.006	1.76	0.56	4.20	1.35	324.43	323.37	1.06	21
D13-07	D13-07	D13-08	320.83	315.84	329.00	8.17	15	392	0.013	1.77	0.38	4.22	0.89	321.75	322.08	0.00	21
D13-08	D13-08	D13-09	315.84	311.29	326.00	10.16	15	390	0.012	1.78	0.39	4.27	0.94	316.81	317.09	0.00	21
D13-09	D13-09	D13-10	311.04	310.18	324.00	12.96	18	399	0.002	1.79	0.57	4.28	1.35	314.85	312.54	2.31	21
D13-10	D13-10	D13-22	310.18	310.10	322.00	11.82	18	67	0.001	1.79	0.76	4.32	1.84	313.26	311.68	1.58	21
D13-22	D13-22	D12-01	310.10	309.30	322.00	11.90	18	344	0.002	1.80	0.55	4.34	1.32	312.99	311.60	1.39	21
D12-01	D12-01	D12-02	309.30	308.42	320.00	10.70	18	406	0.002	1.80	0.57	4.35	1.37	311.59	310.80	0.79	21
D12-02	D12-02	D12-13	308.42	307.42	318.00	9.58	18	95	0.011	1.81	0.26	4.35	0.62	309.28	309.92	0.00	21
D12-13	D12-13	D12-03	307.42	304.38	318.00	10.58	18	309	0.01	1.82	0.27	4.38	0.65	308.30	308.92	0.00	21
D12-03	D12-03	D12-04	304.38	303.38	316.00	11.62	18	402	0.002	1.81	0.53	4.40	1.30	308.00	305.88	2.12	21
D12-04	D12-04	D12-05	303.38	302.54	314.00	10.62	18	336	0.002	1.82	0.54	4.41	1.30	306.31	304.88	1.43	21
D12-05	D12-05	D12-06	302.54	302.05	312.00	9.46	18	190	0.003	1.83	0.53	4.43	1.28	304.89	304.04	0.85	21
D12-06	D12-06	D12-21	302.05	301.69	310.00	7.95	18	145	0.002	1.83	0.54	4.44	1.31	304.08	303.55	0.53	21
D12-07	D12-07	D12-08	301.30	299.97	311.00	9.70	18	243	0.005	1.83	0.36	4.44	0.88	302.39	302.80	0.00	21
D12-08	D12-08	D12-09	299.97	299.60	309.00	9.03	18	60	0.006	1.83	0.34	4.44	0.83	301.01	301.47	0.00	21
D12-09	D12-09	D12-10	299.60	299.06	308.00	8.40	18	96	0.006	1.83	0.36	4.45	0.87	300.68	301.10	0.00	21
D12-10	D12-10	D12-11	299.06	295.27	306.00	6.94	18	330	0.011	1.83	0.25	4.45	0.61	299.91	300.56	0.00	21
D12-11	D12-11	D12-12	295.17	287.58	306.00	10.83	18	102	0.075	1.83	0.10	4.47	0.24	295.67	296.67	0.00	21
D12-12	D12-12	D11-01	287.48	286.50	304.00	16.52	18	386	0.003	1.83	0.53	4.47	1.30	293.56	288.98	4.58	21
D12-21	D12-21	D12-07	301.69	301.30	312.00	10.31	18	155	0.003	1.83	0.54	4.43	1.30	303.46	303.19	0.27	21
D11-01	D11-01	D11-02	286.50	285.62	294.00	7.50	18	265	0.003	1.83	0.47	4.47	1.14	291.91	288.00	3.91	21
D11-02	D11-02	D11-03	285.52	285.15	299.00	13.48	18	162	0.002	1.83	0.56	4.48	1.38	290.79	287.02	3.77	21
D11-03	D11-03	D11-04	285.15	284.18	299.00	13.85	18	385	0.003	1.83	0.54	4.48	1.31	290.09	286.65	3.44	21
D11-04	D11-04	D11-05	284.18	283.49	289.00	4.82	18	184	0.004	1.83	0.44	4.47	1.07	288.44	285.68	2.76	21
D11-05	D11-05	D11-06	283.49	282.78	292.00	8.51	18	230	0.003	1.83	0.48	4.47	1.18	287.65	284.99	2.65	21
D11-06	D11-06	D11-07	282.78	281.94	296.00	13.22	18	243	0.003	1.82	0.46	4.47	1.12	286.65	284.28	2.37	21
D11-07	D11-07	D11-08	281.94	281.10	296.00	14.06	18	242	0.003	1.82	0.45	4.47	1.12	285.60	283.44	2.16	21
D11-08	D11-08	D11-09	281.00	280.05	294.00	13.00	18	376	0.003	1.81	0.53	4.49	1.31	284.55	282.50	2.05	21
D11-09	D11-09	D11-10	280.05	279.32	291.00	10.95	18	293	0.002	1.81	0.53	4.51	1.33	282.90	281.55	1.35	21
D11-10	D11-10	D11-11	279.32	278.32	287.00	7.68	18	404	0.002	1.82	0.54	4.52	1.33	281.61	280.82	0.79	21
D11-11	D11-11	D11-12	278.32	268.45	292.00	13.68	18	251	0.039	1.82	0.14	4.55	0.34	278.92	279.82	0.00	21
D11-12	D11-12	D11-13	268.45	265.31	281.00	12.55	18	309	0.01	1.83	0.27	4.56	0.66	269.34	269.95	0.00	21
D11-13	D11-13	D10-63	265.31	258.11	274.00	8.69	18	304	0.024	1.86	0.18	4.61	0.44	266.01	266.81	0.00	21
D10-63	D10-63	D10-09	258.11	252.18	272.00	13.89	18	250	0.024	1.86	0.18	4.62	0.44	258.81	259.61	0.00	21
D10-09	D10-09	D10-10	252.08	251.00	261.00	8.92	18	404	0.003	1.86	0.53	4.63	1.32	254.97	253.58	1.39	21
D10-10	D10-10	D10-54	250.90	249.99	264.00	13.10	18	340	0.003	1.86	0.53	4.65	1.32	253.09	252.40	0.69	21
D10-54	D10-54	D10-11	249.99	245.00	262.00	12.01	18	128	0.039	1.86	0.14	4.66	0.35	250.60	251.49	0.00	21
D10-11	D10-11	D10-95	244.95	237.09	257.00	12.05	18	309	0.025	1.87	0.17	4.68	0.43	245.64	246.45	0.00	21
D10-95	D10-95	C10-28	237.09	235.99	248.00	10.91	18	256	0.004	1.87	0.42	4.70	1.05	240.48	238.59	1.89	21
C10-28	C10-28	C10-02	235.99	235.56	244.00	8.01	18	101	0.004	1.87	0.42	4.70	1.06	239.26	237.49	1.77	21

SPWA Sewer Evaluation

Pipe Capacity Analysis

Project B1 - Cont.

Pipe ID	Upstream Manhole	Downstream Manhole	Upstream Invert	Downstream Invert	Upstream Manhole Rim Elevation	Upstream Manhole Depth (ft)	Existing Diameter (inches)	Length (feet)	Slope	Existing PWWF (MGD)	Existing PWWF q/Q	Future PWWF (MGD)	Future PWWF q/Q	Manhole HGL Elevation (ft)	Crown Elevation (ft)	Surcharging in Manhole (ft)	Improved Diameter (inches)
C10-02	C10-02	C10-03	235.56	234.56	246.00	10.44	18	356	0.003	1.88	0.52	4.72	1.31	238.77	237.06	1.71	21
C10-03	C10-03	C10-04	234.46	233.75	247.00	12.54	18	256	0.003	1.92	0.53	4.76	1.33	237.05	235.96	1.09	21
C10-04	C10-04	C10-05	233.75	233.03	245.00	11.25	18	256	0.003	1.92	0.53	4.77	1.32	235.79	235.25	0.54	21
C10-05	C10-05	C10-24	232.93	228.15	242.00	9.07	18	95	0.05	1.91	0.13	4.77	0.31	233.51	234.43	0.00	21
C10-24	C10-24	C10-06	228.15	220.31	243.00	14.85	18	162	0.048	1.91	0.13	4.77	0.32	228.73	229.65	0.00	21
C10-06	C10-06	C10-07	220.21	217.14	228.00	7.79	18	333	0.009	1.92	0.29	4.78	0.73	224.85	221.71	3.14	21
C10-07	C10-07	C9-01	217.04	213.10	226.00	8.96	18	333	0.012	1.92	0.26	4.79	0.65	223.20	218.54	4.66	21
C9-01	C9-01	C9-02	213.10	209.52	222.00	8.90	18	307	0.012	1.92	0.26	4.81	0.65	221.54	214.60	6.94	21
C9-02	C9-02	B08-042	209.42	208.50	220.00	10.58	18	317	0.003	1.92	0.52	4.82	1.31	220.00	210.92	9.08	24

Total Length 17,839

SPWA Sewer Evaluation

Pipe Capacity Analysis

Project B2

Pipe ID	Upstream Manhole	Placer County Upstream Manhole[1]	Downstream Manhole	Placer County Downstream Manhole[1]	Upstream Invert	Downstream Invert	Upstream Manhole Rim Elevation	Upstream Manhole Depth (ft)	Existing Diameter (inches)	Length (feet)	Slope	Existing PWWF (MGD)	Existing PWWF q/Q	Future PWWF (MGD)	Future PWWF q/Q	Manhole HGL Elevation (ft)	Crown Elevation (ft)	Surcharging in Manhole (ft)	Improved Diameter (inches)
11825	B08-042	C9-03	B08-040	C9-M5	210.41	210.26	233.52	23.12	15	52	0.003	3.03	1.37	8.07	3.65	226.03	211.66	14.37	24
11826	B08-040	C9-M5	B08-045	-	210.26	209.91	233.23	22.97	15	102	0.003	3.03	1.24	8.07	3.30	224.08	211.51	12.57	24
11827	B08-045	-	B08-047	-	209.91	209.03	231.26	21.35	21	205	0.004	3.73	0.55	9.05	1.34	220.27	211.66	8.60	24
11828	B08-047	-	B08-044	-	209.03	207.38	228.43	19.40	21	498	0.003	3.72	0.63	9.04	1.53	218.67	210.78	7.88	24
11829	B08-044	-	B08-037	-	207.38	204.63	232.44	25.05	21	402	0.007	3.71	0.44	9.06	1.07	214.78	209.13	5.64	24
11830	B08-037	-	B08-043	-	204.63	203.26	227.37	22.74	21	342	0.004	3.84	0.59	9.26	1.42	211.63	206.38	5.25	24
11831	B08-043	-	B07-108	-	203.26	201.42	232.44	29.17	21	461	0.004	3.83	0.59	9.24	1.43	208.83	205.01	3.82	24
12891	B07-108	-	B07-107	-	201.42	199.59	228.49	27.06	21	461	0.004	3.82	0.59	9.21	1.42	205.07	203.17	1.90	24
12892	B07-107	-	B07-405	-	199.59	196.50	225.56	25.98	21	324	0.01	3.81	0.38	9.19	0.92	200.90	201.34	0.00	24
									Total Length	**2,846**									

1. Placer County Manhole IDs are based on County GIS. Corresponding Placer County manhole IDs are estimated and field verification may be required.

SPWA Sewer Evaluation

Pipe Capacity Analysis

Project C

Pipe ID	Upstream Manhole	Placer County Upstream Manhole[1]	Downstream Manhole	Placer County Downstream Manhole[1]	Upstream Invert	Downstream Invert	Upstream Manhole Rim Elevation	Upstream Manhole Depth (ft)	Existing Diameter (inches)	Length (feet)	Slope	Existing PWWF (MGD)	Existing PWWF q/Q	Future PWWF (MGD)	Future PWWF q/Q	Manhole HGL Elevation (ft)	Crown Elevation (ft)	Surcharging in Manhole (ft)	Improved Diameter (inches)
E9-09	E9-09	E9-09	E9-10	E9-10	242.62	242.04	253.00	10.38	15	360	0.002	0.59	0.35	3.58	2.13	253.00	243.87	9.13	21
E9-10	E9-10	E9-10	E9-11	E9-11	242.04	241.46	256.00	13.96	15	360	0.002	0.67	0.40	3.58	2.13	255.64	243.29	12.35	21
E9-11	E9-11	E9-11	E9-12	E9-12	241.46	240.88	262.00	20.54	15	363	0.002	0.68	0.41	3.52	2.10	253.00	242.71	10.29	21
E9-12	E9-12	E9-12	E9-13	E9-13	240.88	240.30	262.00	21.12	15	356	0.002	0.68	0.40	3.47	2.06	250.42	242.13	8.29	21
E9-13	E9-13	E9-13	E9-14	E9-14	240.20	240.02	264.00	23.80	15	93	0.002	0.67	0.36	3.40	1.85	247.95	241.45	6.50	21
E9-14	E9-14	E9-14	D9-01	D9-01	239.92	239.36	263.00	23.08	15	345	0.002	0.79	0.47	3.60	2.13	247.34	241.17	6.17	21
D9-01	D9-01	D9-01	D9-02	D9-02	239.36	238.80	259.00	19.64	15	324	0.002	0.80	0.46	3.54	2.03	244.78	240.61	4.16	21
D9-02	D9-02	D9-02	D9-03	D9-03	238.70	238.59	253.00	14.30	15	55	0.002	0.80	0.43	3.49	1.87	242.45	239.95	2.50	21
D9-03	D9-03	D9-03	D9-25	D9-25	238.49	238.10	251.00	12.51	15	211	0.002	0.95	0.53	3.72	2.07	242.06	239.74	2.32	21
D9-25	D9-25	D9-25	D9-04	D9-04	238.10	237.90	247.00	8.90	15	154	0.001	1.00	0.66	3.74	2.48	240.39	239.35	1.04	21
D9-04	D9-04	D9-04	D9-05	D9-05	237.89	233.08	246.00	8.11	15	331	0.015	1.06	0.21	3.78	0.75	238.70	239.14	0.00	21
D9-05	D9-05	D9-05	D9-06	D9-06	233.08	232.35	237.00	3.92	15	134	0.005	1.10	0.36	3.80	1.23	237.00	234.33	2.67	21
D9-06	D9-06	D9-06	D9-07	D9-07	232.25	231.76	236.00	3.75	15	266	0.002	1.12	0.62	3.84	2.14	236.00	233.50	2.50	21
D9-07	D9-07	D9-07	D9-08	D9-08	231.76	231.21	242.00	10.24	15	304	0.002	1.12	0.63	3.79	2.13	237.74	233.01	4.73	21
D9-08	D9-08	D9-08	D9-09	D9-09	231.11	230.53	236.00	4.89	15	329	0.002	1.11	0.63	3.75	2.13	236.00	232.36	3.64	21
D9-09	D9-09	D9-09	B08-012	B08-012	230.43	230.25	235.00	4.57	15	97	0.002	1.12	0.62	3.72	2.06	235.00	231.68	3.32	21
11819	B08-012	D9-10	B08-015	D9-11	227.80	227.74	249.99	22.19	15	345	0.0002	1.16	2.12	3.77	6.92	236.01	229.05	6.97	21
11821	B08-015	D9-11	B08-023	D9-12	227.74	225.14	262.50	34.76	15	306	0.008	1.09	0.28	3.53	0.91	233.94	228.99	4.95	21
11822	B08-023	D9-12	B08-208	D9-43	225.14	222.68	250.89	25.75	15	289	0.009	1.08	0.28	3.50	0.91	232.02	226.39	5.63	21
9353385	B08-208	D9-43	B08-031	D9-13	222.68	220.50	250.03	27.35	15	111	0.02	1.15	0.20	3.55	0.61	230.19	223.93	6.26	21
11823	B08-031	D9-13	B08-041	C9-04	220.50	210.68	244.21	23.71	15	364	0.027	1.14	0.17	3.54	0.52	229.45	221.75	7.70	21
11824	B08-041	C9-04	B08-042	C9-03	210.68	210.41	235.39	24.71	15	150	0.002	1.14	0.64	3.53	1.99	227.03	211.93	15.10	21
									Total Length	**5,647**									

1. Placer County Manhole IDs are based on County GIS. Corresponding Placer County manhole IDs are estimated and field verification may be required.

SPWA Sewer Evaluation

Pipe Capacity Analysis

Project E

Pipe ID	Upstream Manhole	Downstream Manhole	Upstream Invert	Downstream Invert	Upstream Manhole Rim Elevation	Upstream Manhole Depth (ft)	Existing Diameter (inches)	Length (feet)	Slope	Existing PWWF (MGD)	Existing PWWF q/Q	Future PWWF (MGD)	Future PWWF q/Q	Manhole HGL Elevation (ft)	Crown Elevation (ft)	Surcharging in Manhole (ft)	Improved Diameter (inches)
16606	D03-100	D03-099	119.80	118.90	138.94	19.15	15	298	0.003	3.11	1.36	3.43	1.50	127.26	121.05	6.21	21
16607	D03-099	D03-086	118.90	118.59	137.50	18.60	15	104	0.003	3.11	1.36	3.43	1.50	125.25	120.15	5.10	21
16604	D03-086	D03-072	118.59	118.00	136.97	18.38	18	294	0.002	3.11	1.02	3.49	1.15	124.55	120.09	4.46	21
16599	D03-072	D03-069	118.00	117.64	137.50	19.50	18	263	0.001	3.11	1.22	3.48	1.37	123.77	119.50	4.27	21
2957	D03-069	D03-067	117.64	117.07	142.50	24.86	18	315	0.002	3.52	1.22	3.89	1.35	123.08	119.14	3.94	21
2844	D03-067	D03-046	117.07	116.78	139.62	22.55	18	159	0.002	3.67	1.27	4.05	1.40	122.05	118.57	3.48	21
2845	D03-046	D03-031	116.78	116.19	140.32	23.54	18	330	0.002	3.67	1.27	4.04	1.40	121.48	118.28	3.20	21
2902	D03-031	D03-030	116.03	115.67	138.41	22.38	18	180	0.002	3.87	1.27	4.25	1.40	120.31	117.53	2.78	21
2903	D03-030	D03-029	115.97	115.17	137.27	21.30	18	400	0.002	3.87	1.27	4.24	1.39	119.60	117.47	2.13	21
2229	D03-029	D02-043	115.17	113.65	132.50	17.33	18	400	0.004	3.85	0.92	4.33	1.03	118.04	116.67	1.37	21
2232	D02-043	D02-048	113.65	112.63	130.48	16.83	18	341	0.003	3.84	1.03	4.32	1.16	116.41	115.15	1.26	21
2234	D02-048	D02-047	112.63	112.25	133.61	20.99	18	125	0.003	3.83	1.03	4.30	1.15	115.03	114.13	0.90	21
2200	D02-047	D02-046	112.25	111.58	134.09	21.84	18	225	0.003	3.95	1.06	4.42	1.19	114.52	113.75	0.77	21
2202	D02-046	D02-045	111.58	110.38	135.67	24.09	18	400	0.003	3.94	1.06	4.41	1.18	113.57	113.08	0.49	21
17110	D02-045	D02-183	108.53	108.47	134.62	26.09	18	50	0.001	4.59	1.94	5.01	2.12	110.24	110.03	0.21	21
							Total Length	**3,883**									

Technical Memorandum

South Placer Regional Wastewater & Recycled Water Systems Evaluation Project

Subject: Wastewater Treatment Plants Expansion Requirements (TM 4b)

Prepared For: Art O'Brien – City of Roseville

Prepared by: Marilyn Bailey – RMC
Austin Peterson – RMC
Lea Fisher

Reviewed by: Dave Richardson – RMC

Date: March 28, 2006

Reference: 0091-004

1 Introduction

The South Placer Wastewater Authority (SPWA) is evaluating its wastewater systems to determine the effects of infill development and conversion of agricultural and open land to residential and commercial development. As part of this effort, the additional treatment plant flows and loadings from the projected growth have been analyzed in the following Technical Memoranda:

- *Dry Weather Flow Projection for the Ultimate SPWA Service Area (Including Urban Growth Areas) TM No.2b* dated *November 4, 2005,* which developed new unit flow factors and Average Dry Weather Flow Projections
- *Wet Weather Flow Projection for the Ultimate SPWA Service Area (Including Urban Growth Areas) TM No.2c dated December 1, 2005* which developed wet weather flow projections and used a hydraulic model for assessment of the collection system and peak flows into the treatment plants.
- *WWTP Projected Loadings and Buildout (TM 4a) dated December 9, 2005,* which analyzed flow distribution from the service area to the two WWTPs, developed a timeline for service area buildout, and projected future BOD and suspended solids loadings to the plants

This technical memorandum (TM 4b) establishes flow and loading peaking factors, develops facility expansion recommendations to handle the projected flows and loadings at buildout, and presents a timeline for phasing the construction of the improvements.

TM 4b is organized as follows:

1. Introduction
2. Flows and Loadings
3. Dry Creek Expansion Recommendations
4. Pleasant Grove Expansion Recommendations
5. Construction Phasing and Estimated Costs

2 Flows and Loadings

2.1 Average Dry Weather Flows

Average dry weather flows (ADWF) for projected buildout of the SPWA service area were developed in TM 2b as follows; Dry Creek WWTP at 19.3 mgd and Pleasant Grove WWTP at 23.4 mgd. The SPWA has historically used 3 mgd increments of flow in its long-term planning efforts. That convention will also be followed for this TM. Therefore, the ADWF to be used for long-term master planning will be rounded up to the nearest 3 mgd increment, or 21 mgd for Dry Creek and 24 mgd for Pleasant Grove. Since most of the growth in the SPWA will be in the Pleasant Grove service area, an extra degree of conservatism is warranted there when allocating space for new facilities. Therefore, the layout for new facilities at Pleasant Grove will also show the requirements for an ultimate ADWF of 27 mgd.

This TM uses the following ADWF's for evaluation of the wastewater systems.

Table 1 ADWF for System Evaluation

Plant	ADWF (mgd)
Dry Creek	21 (buildout)
Pleasant Grove	24 (buildout) 27 (ultimate)

2.2 Flow Peaking Factors

While the ADWF is usually thought of as the rated capacity of a treatment plant, the design of treatment systems must also accommodate significant variations in influent flow. A treatment plant must be designed to prevent hydraulic overloads and wash out of solids during peak day and peak hour events. It must also be able to meet discharge limits during the sustained higher flows experienced during the peak month of wet weather. This section discusses the flow peaking factors that will be used in the evaluation of both plants.

2.2.1 Peak Day and Peak Hour

As part of the modeling effort for the trunk sewers, Brown and Caldwell developed peak wet weather flow hydrographs. The hydrographs, which are discussed in more detail in TM 2c, show the estimated peak day and peak hour flows to each treatment plant at buildout. Table 2 summarizes the results of the peak wet weather modeling.

Table 2 –Estimated Peak Wet Weather Flows from Wet Weather Model

Plant	Peak Day		Peak Hour	
	Flow (mgd)	Peaking Factor [1]	Flow (mgd)	Peaking Factor [1]
Dry Creek	43.5	2.3	56.5	2.9
Pleasant Grove	34.2	1.5	46.4	2.0

[1] Peaking Factors based on the ADWF at buildout from the model, i.e. 19.3 mgd for Dry Creek and 23.4 mgd for Pleasant Grove

The peaking factors that have been historically used at Dry Creek are 2.5 for peak day and 3.0 for peak hour. The wet weather model may not fully account for continued degradation of existing sewers and I/I from new sewers as they age. Therefore, to provide a safety factor, the historical peaking factors of 2.5 and 3.0 will continue to be used for planning purposes for Dry Creek. For Pleasant Grove, lower peaking factors are appropriate since the sewers are newer. The model estimated peaking factors of 1.5 and 2.0. Adding a safety factor to the model predictions for planning purposes, the resulting peaking factors for Pleasant Grove are 2.0 for peak day and 2.5 for peak hour.

2.2.2 Peak Month

The maximum monthly flow peaking factor was determined by calculating a 30-day running average of the daily measured flows at the plant dating back to fiscal year 2000/2001and comparing the maximum monthly flow to the average dry weather flow for the same year. The maximum peak month factor for Dry Creek was 1.39 in 2003/2004. Rounding up, a peak month flow factor of 1.4 will be used in the system evaluation. The operating period of Pleasant Grove has been too short to establish a meaningful peak month factor, so the same factor of 1.4 will be used for the evaluation of Pleasant Grove.

2.3 Projected Plant Influent Flows

Table 3 summarizes the plant influent flows that will be used for sizing expansion requirements at the two plants.

Table 3 – Plant Influent Flows for System Evaluation

Plant	ADWF (mgd)	Peak Month		Peak Day		Peak Hour	
		Peaking Factor	Flow (mgd)	Peaking Factor	Flow (mgd)	Peaking Factor	Flow (mgd)
Dry Creek	21	1.4	29.4	2.5	52.5	3.0	63
Pleasant Grove (buildout)	24	1.4	33.6	2.0	48	2.5	60
Pleasant Grove (ultimate)	27	1.4	37.8	2.0	54	2.5	67.5

2.4 Estimated Loadings

TM 4a analyzed the biochemical oxygen demand (BOD) and total suspended solids (TSS) data from the past five years to determine the influent loading to each plant. The dry weather BOD and TSS concentrations determined in TM 4a were as follows:

- Dry Creek: BOD 275 mg/l and TSS 310 mg/l
- Pleasant Grove: BOD 285 mg/l and TSS 340 mg/l

The maximum monthly BOD and TSS loadings were compared to average annual loadings to determine a peaking factor for the peak month at each WWTP. Average annual loadings were calculated by averaging the daily measured loading for each fiscal year at the plant. The data used in the analysis for Dry Creek dates back to fiscal year 2000/2001. Data for Pleasant Grove is only available since the plant began operating in 2004. The plant experienced an initial phase in period between June 2004 and January 2005 during which time the flow rate was incrementally increased. Since February 2004 the plant has been treating approximately 6.6 mgd and the loading values used in this analysis are based on data collected since that time.

The maximum monthly loading was found by calculating a 30-day running average of the daily measured loadings at the plant and selecting the highest value. The peak month factor for Dry Creek ranged from 1.17 to 1.45 with an average peak month factor of 1.2 for both BOD and TSS. The peak month factor for Pleasant Grove was only 1.1. However, with such a short operating history to draw from, it is more realistic to expect that the peaking factors at Pleasant Grove over time will be similar to that of Dry Creek, i.e. a peak month factor of 1.2.

Table 4 summarizes the projected BOD and TSS loadings at the two treatment plants.

Table 4 Projected Influent Loadings

Plant	ADWF	BOD		TSS	
	(mgd)	(mg/l)	(lb/day)	(mg/l)	(lb/day)
Dry Creek	21	275	48,200	310	54,300
Maximum Month Factor		1.2		1.2	
Pleasant Grove (buildout)	24	285	57,000	340	68,100
Pleasant Grove (ultimate)	27	285	64,200	340	76,600
Maximum Month Factor		1.2		1.2	

2.5 Design Flows and Loadings

Table 5 summarizes the plant influent flows and loadings that will be used for the wastewater systems evaluation.

Table 5 Projected Influent Flows and Loadings

	Dry Creek	Pleasant Grove (buildout)	Pleasant Grove (ultimate)
Flow (mgd)			
Average Dry Weather Flow (ADWF)	21.0	24	27
Peak Month Flow (PMF)	29.4	33.6	37.8
Peak Day Wet Weather Flow (PDWWF)	52.5	48.0	54.0
Peak Hour Wet Weather Flow (PHWWF)	63.0	60.0	67.5
BOD Loading (lb/day)			
Average	48,200	57,000	64,200
Maximum Month	57,800	68,400	77,000
TSS Loading (lb/day)			
Average	54,300	68,100	76,600
Maximum Month	65,200	81,700	91,900

3 Dry Creek Expansion Recommendations

The Dry Creek WWTP liquid treatment processes include screening and grit removal, primary treatment, secondary treatment including denitrification, effluent filtration, and disinfection with gaseous chlorine (soon to be replaced with UV disinfection). A portion of the effluent passes through cooling units to keep the blended effluent within the required temperature range for discharge. The solids handling process includes gravity belt thickeners, anaerobic digesters, and belt press dewatering.

The Dry Creek plant, originally constructed in 1974, was expanded in 1991 to treat an ADWF of 18 mgd. Beginning in June of 2004 part of the flow has been diverted to the new Pleasant Grove plant, so the current ADWF to Dry Creek is only 10.5 mgd, or 58 percent of its design flow.

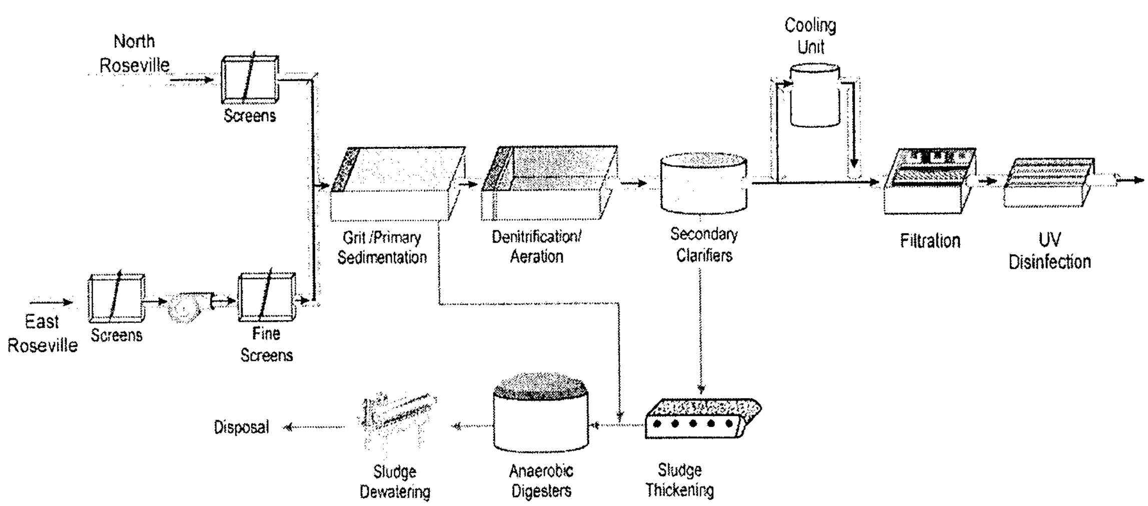
Equally as important as the hydraulic capacity of a plant is its organic treatment capacity. The 1991 expansion of the plant was based on an influent BOD concentration of 160 mg/l for an average BOD loading of 24,000 lb/day. As discussed earlier in this TM, the concentrations of influent BOD has significantly increased since 1991. Since the completion of the diversion of flow to Pleasant Grove in February 2005, the average BOD loading to the Dry Creek plant has been 21,500 lb/day, or 89 percent of its design BOD capacity.

The recommended expansion requirements are driven by four factors:

- Flow capacity to meet the anticipated ADWF of 21 mgd and peak hydraulic flow of 63 mgd.
- Organic treatment capacity to meet an anticipated BOD loading of 48,200 lb/day and a TSS loading of 54,300 lb/day
- Denitrification to meet an expected new NPDES limit of 10 mg/l-N
- Replacement of older, under-performing facilities

The proposed treatment process for the system expansion is similar to that of the existing plant, with the exception of the addition of fine screens, and is illustrated below on Figure 1.

Figure 1 Dry Creek Treatment Schematic



The recommended facilities are summarized in Table 26 – Dry Creek Design Criteria and the proposed layout for the new facilities is shown on Figure 2. Both are included at the end of this TM. The design basis for each process is discussed below.

3.1 Headworks

Wastewater enters the Dry Creek plant via the East Roseville gravity sewer and the North Roseville force main. The East Roseville flow is screened and pumped at the plant, and was sized in 1991 for an ADWF of 12 mgd. The North Roseville force main has a separate screening facility at the plant and is sized for an ADWF of 6 mgd. Most of the flow diverted to the new Pleasant Grove treatment plant came from the North Roseville force main, so the flow currently entering the plant through the force main is small.

Routing of trunk sewers for the new Urban Growth Areas has not been finalized as of this writing. For the purposes of this analysis it is assumed that up to 6 mgd of dry weather flow, including flows from Placer Vineyards and potentially West Dry Creek, will enter the plant through the North Roseville force main. The remaining 15 mgd of dry weather flow will enter through the influent pump station. This assumption should be revisited as the routing of the trunk sewers serving the new UGA's is better established.

3.1.1 Coarse Screens

The existing screening facility for the East Roseville trunk sewer includes 2 mechanically cleaned bar screens with ¾ inch openings and a manually cleaned bypass screen. There is an existing spare channel. The system expansion assumes the addition of one mechanically cleaned bar screen in the existing spare channel.

3.1.2 Influent Pump Station

The existing East Roseville influent pump station consists of 7 pumps which are divided between two pump stations. Each of the 75 HP pumps is sized for 3,000 gpm giving a firm pumping capacity with one pump out of service of 26 mgd.

The influent pump station should be designed to pump a projected PHWWF of 45 mgd, which corresponds to the ADWF of 15 mgd entering the plant through the pump station. Variable speed pumps are needed to accommodate the range of flows expected, from an existing diurnal minimum dry weather flow of about 5 mgd to a projected peak hour flow of 45 mgd. The system expansion includes a new influent pump station would contain 5 variable speed pumps, 4 duty and 1 standby. Assuming the same 60 feet of head as the existing pump station, each pump would be sized for a maximum capacity of 8,000 gpm (11.5 mgd) and a motor size of 175 HP. During dry weather flow, 1 to 3 pumps would operate to cover the diurnal variation of dry weather flows. For the peak hour flows of 45 mgd, 4 pumps would operate with the fifth pump as standby. The pumps would be housed in a new influent pump station to the west of the existing screenings facility.

3.1.3 Fine Screens

The Dry Creek plant does not currently have fine screens. The only screens are the mechanically screened bar screens with ¾ inch openings. The coarse screens allow plastics to pass through to downstream treatment processes. The plastics accumulate in the digesters and detrimentally impact the quality of the digested sludge. Addition of a fine screen process is proposed to remove the fine plastics from the treatment process. For layout and cost estimating purposes, the fine screens are assumed to be part of the new influent pump station. During predesign, consideration should also be given to locating the fine screens downstream of the influent pump station near the grit basins so that the screened materials could be handled above grade and so that a single fine screening facility would handle flows from both the influent pump station and the North Roseville force main.

The fine screens are sized to pass the PHWWF of 45 mgd entering the plant via the pump station. For layout and cost estimating purposes this evaluation assumes the addition of three 6 mm (1/4") band screens, each with a capacity of 15 mgd. The facility would also include odor control and a screenings washer/compactor to remove organics caught by the screens.

3.2 Grit Removal

There are two existing aerated grit basins each 640 square feet and a volume of 6400 cubic feet. At the 1991 design ADWF of 18 mgd, the overflow rate is 9.8 gpm/sf and the detention time is 7.7 minutes.

Sizing of new grit removal basins would also be based on a hydraulic loading rate. However, the in-plant recycle streams, such as such as filter backwash and the filtrate from the solids thickening and dewatering processes, are returned to the main plant flow upstream of the grit basins. For Dry Creek, this adds an additional 5 to 9 percent of flow, so the effective flow to the grit basins and downstream processes is higher than the influent flow to the plant. The estimated flow rates for the recycle streams and the resulting flow rates for process sizing are shown in the following table:

Table 6 Dry Creek Flow to Processes, Including Recycle Streams

		ADWF	Peak Month	PDWWF	PHWWF
Influent Flow	mgd	21.0	29.4	52.5	63.0
Recycle Flows					
Spray water and washdown	mgd	0.2	0.3	0.3	0.3
Backwash Recycle	mgd	0.6	1.4	1.9	1.9
Dewatering Recycle	mgd	0.3	0.4	0.4	0.4
GBT Recycle	mgd	0.5	0.6	0.6	0.5
Total Recycle Flows	mgd	1.5	2.6	3.1	3.1
Percent of Influent Flow	%	7.2%	9.0%	5.9%	4.9%
Total Flow with Recycle	mgd	**22.5**	**32.0**	**55.6**	**66.1**

One additional aerated grit basin similar in size and configuration to the existing units is recommended. . Three aerated grit basins would have a total surface area of 1,920 square feet and a volume of 19,200 cubic feet. The resulting design criteria for three aerated grit basins would be as follows:

Table 7 Dry Creek Grit Basin Sizing Criteria

Grit Basin Criteria	ADWF (22.5 mgd)	Peak Day (55.6 mgd)
Overflow Rate, gpm/sf	8.2	20.2
Detention Time, minutes	9.2	3.7

Addition of a third grit basin will also require one additional grit blower, grit pump, grit classifier, and grit cyclone similar in size and configuration to the two existing units of each type.

3.3 Primary Sedimentation

There are four existing primary sedimentation basins. Each is 225 feet long, 20 feet wide, and 10 feet deep. The current design surface overflow rate is 1,000 gpd/sf (at ADWF of 18 mgd) and 2,500 gpd/sf (at PDWWF of 45 mgd).

Sizing new primary sedimentation basins would be based on a hydraulic surface loading rate using the effective flow rates, including recycle streams, shown in Table 6. The grit and sedimentation basins are constructed with one grit basin serving two primary sedimentation basins. Keeping this configuration would mean adding the primary sedimentation basins in multiples of two. The following table shows the design surface loading rates and the actual overflow rates resulting from adding two or four additional sedimentation basins.

Table 8 Dry Creek Primary Sedimentation Sizing

		ADWF	Peak Month	PDWWF	PHWWF
Flow Rate	mgd	22.50	32.00	55.60	66.10
Maximum Design Overflow Rate	gpd/sf	1,000	1,200	1,600	2,500
Total Area Required	sf	22,500	26,667	34,750	26,440
Existing Area	sf	18,000	18,000	18,000	18,000
Additional Area Required	sf	4,500	8,667	16,750	8,440
Additional Basins Needed	ea	1.0	2.0	4.0	2.0
Actual overflow rate w/ 6 basins	gpd/sf	820	1,160	2,010	2,390

Adding two new sedimentation basins would meet the design overflow rates for all conditions except for the peak day wet weather flow, although the process would be operating near its maximum recommended capacity for the peak month during wet weather. The recommendation is therefore to add two new primary sedimentation basins, but to provide space for two additional units should future operating experience at the plant indicate that a lower overflow rate during wet weather is desirable.

The two new primary sedimentation basins would require the addition of three primary sludge pumps and one primary scum pump. The new sedimentation basins and the additional grit basin would be located to the north of the existing basins.

3.4 Odor Control

The new influent pump station, fine screens, grit basin, and primary sedimentation basins would be covered for odor control. The recommended odor control system for the new processes is soil bed biofilters located near or adjacent to the new process structures.

3.5 Secondary Treatment Process

The secondary aeration configuration at Dry Creek includes denitrification basins ahead of the secondary aeration basins. There are eight existing aeration basins; four from the original plant design in 1974 and four from the 1991 expansion. There are six secondary clarifiers, four from the original plant design and two from the 1991 expansion. The aeration basins and clarifiers from the original 1974 plant are smaller, shallower, and less efficient than the corresponding basins from 1991. For all of the secondary treatment alternatives, it is assumed that the older aeration basins and clarifiers will be replaced with new units. The development of expansion alternatives therefore considers only the 1991 basins as existing.

The secondary treatment processes, including the denitrification basins, aeration basins and the secondary clarifiers, are affected by both hydraulic and the organic loading rates. Aeration basins are sized to maintain a certain inventory (pounds) of microorganisms to provide the desired solids residence time. Sizing the secondary treatment process is a balance between the aeration basin volume versus the solids loading to the clarifiers. For example, operating the plant at a high mixed liquor concentration in the aeration basins reduces the needed volume of the aeration basins, but results in a high solids loading to the clarifiers which, in turn, must be sized larger to accommodate the solids loading. Conversely, a low mixed liquor concentration increases the required volume in the aeration basins but reduces the required clarifier area since the solids loading would be lower.

The sizing criteria discussed in the following sections is based on modification of the sizing used in the 1991 expansion to accommodate the higher influent BOD loading currently experienced, and review of the plant operation during the heavy storms of early January 2006 in which the plant influent flow was 28 to 30 mgd. During that storm, everything in the plant was on line except the old mechanical aeration basins. There was some carryover of solids from the secondary clarifiers. Effectively, the plant was pushed to the limit but was able to meet permit limits due to good work by the operators.

The design of the aeration basins for the system expansion is based on producing an effluent with a total nitrogen concentration of less than 10 mg/l-N with a mean cell residence time (MCRT) of 11 days during the summer and 13 days during the winter. The governing criteria for the aeration basin sizing is the peak month, wet weather condition with an MCRT of 13 days. For the clarifiers at Dry Creek, the peak day loading conditions were the governing conditions.

Two options for sizing the aeration basins and clarifiers were considered. The first option uses a mixed liquor suspended solids (MLSS) concentration of 2,500 mg/l in the summer and 3,000 mg/l in the winter. The second option uses a MLSS of 3,000 mg/in the summer and 3,500 mg/in the winter. The following table summarizes the process sizing for the two options. For the clarifiers, as shown in Table 9, the governing sizing criteria at the lower MLSS concentration is the hydraulic loading and, for the higher MLSS concentration, it is the solids loading rate.

Table 9 Dry Creek Secondary Treatment Sizing

		Option 1	Option 2
Aeration Basin Sizing			
Governing Condition		Peak Month	Peak Month
Total Sludge Produced	lb/day	31,800	31,800
MCRT Required	days	13	13
Required Solids for MCRT	lb	413,400	413,400
Design MLSS	mg/l	3,000	3,500
Required Basin Volume for MCRT	MG	16.5	14.2
Volume Existing Basins (1)	MG	4.2	4.2
Additional Volume Needed	MG	12.3	10.0
Additional Basins Needed	ea	12	10
Clarifier Sizing			
Governing Condition		PDWWF	PDWWF
Flow	mgd	55.60	66.10
Anticipated Total RAS Flow	mgd	36.00	40.00
MLSS	mg/l	3,000	3,500
Max Hydraulic Overflow Rate	gal/sqft-day	700	700
Maximum Solids Loading Rate	lb/sqft-day	32	32
Req'd Area for Hydraulic Loading	sqft	79,400	94,400
Req'd Area for Solids Loading	sqft	71,600	96,800
Area Existing Clarifiers (1)	sqft	24,500	24,500

		Option 1	Option 2
Additional Area Needed	sqft	54,900	72,300
Additional Clarifiers Needed	ea	5	6
(1) Volume and Area of existing aeration basins and clarifiers include only those added built duirng the 1991 Expansion. For both options it is assumed that the older aeration basins and clarifiers built in 1974 would be replaced.			

The better match between aeration basin and clarifier volume appears to be Option 2, which is designed for the higher MLSS concentration. One more clarifier is needed for this option, but it is less expensive to construct and operate than the two additional aeration basins needed for Option 1. The recommendation is therefore to add 10 additional nitrification/aeration basins for a total of 14 basins. Four of the new aeration basins would be replacements for the older basins from 1974. The remaining six aeration basins would be located north of the 1991 basins. The new basins will be similar in configuration to the 1991 basins except that the anoxic zone, which constitutes about 20 percent of the basin volume, will be physically separated from the aeration zone. This will prevent the backflow between the anoxic and aeration zones that currently diminishes the effectiveness of the anoxic zone. The cost estimate includes an allowance for retrofitting the existing anoxic basins to minimize the cross-circuiting between basins.

The peak air demand at buildout is estimated to be 35,300 scfm. Currently there are three motor driven blower and one engine driven blower, each sized at 8,775 scfm. One additional motor driven blower would be needed to meet the projected air demands with the fifth blower acting as standby. The additional blower would be located in the spare space in the existing blower building.

For the clarifiers, six additional 125 ft diameter clarifiers are needed, for a total of 8 clarifiers. The four smaller clarifiers and RAS pump station from 1974 will be replaced with two new clarifiers and a RAS pump station similar in configuration to the 1991 clarifiers. The remaining four new clarifiers and a new RAS pump station will be located north of the 1991 clarifiers.

3.6 Effluent Cooling

The Dry Creek plant has 4 effluent cooling units with space for an additional 6 units. The projected buildout flow of 21 mgd is approximately 20 percent higher than the original ADWF design of 18 mgd. Assuming the cooing unit sizing increases by the same ratio, one additional cooling unit would be needed. It would be installed in one of the spare spaces.

3.7 Effluent Filtration

There are three existing filters, each with four 347 square foot cells per filter. The total existing surface area is 4,164 sq ft, or 3,817 sq ft with one cell out of service for backwash. They were designed to maintain a filter surface loading of less than 5 gpm/sq ft (Department of Health Services (DHS) guidelines for Title 22 reuse) at 27 mgd (peak hour dry weather flow). The loading rate for the current PDWWF of 45 mgd would be 7.9 gpm/sf with one cell out of service.

The filters must meet the DHS loading criteria of 5 gpm/sf for the peak hour of the ADWF, which is assumed to have a peaking factor of 1.5 over the ADWF. For the system expansion, a more conservative design criteria is recommended. Even though it is unlikely that recycled water would be used for irrigation during the peak wet weather day and thus would not have to meet the 5 gpm/sf loading rate recommended by DHS, the effluent must still meet a turbidity limit of less than 2 ntu. Maintaining the surface loading rate closer to 5 gpm/sf during PDWWF will help the plant meet the effluent turbidity of 2 ntu during wet weather events. The following table shows the design surface loading rates and the actual overflow rates resulting from adding two or three additional filters similar in size and configuration to the existing filters.

Table 10 Dry Creek Filter Loading Rates

		ADWF	Peak Hour of ADWF	Peak Month	PDWWF
Flow Rate	mgd	22.50	33.75	32.00	55.60
Desired Design Overflow Rate	gpm/sf	5	5	5	5
Existing Filter Area (1)	sf	3817	3817	3817	3817
Area per Additional Filter	sf	347	347	347	347
Overflow Rate with 2 new filters	gpm/sf	2.37	3.55	3.37	5.86
Overflow Rate with 4 new filters	gpm/sf	1.96	2.94	2.78	4.84
(1) With one cell out of service for backwashing					

Adding two additional filters would meet the DHW Title 22 criteria for all conditions except peak day wet weather. The recommendation is therefore to add two new filters, but to provide space for an additional unit should future operating experience at the plant indicate that a lower overflow rate during wet weather is desirable. The new filters would be located to the east of the existing filters. New waste backwash pumps and a larger backwash pipe would also be required to handle the additional flow.

3.8 Disinfection

The Dry Creek WWTP is in the process of converting from disinfection with gaseous chlorine to UV disinfection. The proposed UV system is designed for a peak day plant influent flow of 45 mgd and will have five channels, with 4 UV banks per channel. Based on a ratio of the peak day plant influent flow rates (52.5 mgd/45 mgd, or 1.17), one additional UV channel is needed to meet the future flow rates, for a total of 6 channels. The new channel would be built to the east of the five proposed UV channels. The cost estimate assumes that the initial 5 UV channels are already in place.

3.9 Solids Thickening

There are two existing, 2 meter wide gravity belt thickeners. Based on discussion with plant staff, it is assumed that the thickening process would operate 24 hours per day. As shown in the table below, a single belt thickener is adequate to meet the projected thickening requirements under both ADWF and peak month conditions. The second existing belt thickener would function as the redundant unit, or both could be put in service to shorten the thickening time below 24 hours per day. No additional thickening facilities are required.

Table 11 Dry Creek Solids Thickening Sizing

		ADWF	Peak Month
WAS Production	lb/day	27,550	31,800
WAS Production	mgd	0.46	0.49
Hours of Operation	hrs/day	24	24
WAS Flow to Thickeners	gpm	319	340
Maximum GBT Loading Rate	gpm/unit	400	400
Required Number of Thickeners	ea	1	1

3.10 Solids Stabilization

There are two existing anaerobic digesters, each 90 feet in diameter with a volume of 1.18 million gallons. Currently a single digester is used with a volatile solids loading rate of 0.13 lb VS/cf-day and a detention time of about 14.5 days. The second digester acts as a redundant unit. The future digesters were sized for the peak month solids production using two methods. The first replicates the existing loading criteria, i.e. a volatile solids loading rate of 0.13 lb VS/cf-day with a redundant digester available. The second method assumes an overall volatile solids loading rate of 0.10 lb VS/cf-day with no redundant digester. The results of the analysis are shown below.

Table 12 Dry Creek Digester Sizing

		Option 1	Option 2
Peak Month Solids Production	lb/day	72,980	72,980
Peak Month VSS Production	lb/day	57,980	57,980
VSS Design Criteria	lb VSS/cf/day	0.13	0.10
Total Volume Needed	MG	3.3	4.3
Volume per Digester	MG	1.18	1.18
Total Digesters Needed			
Duty	ea	3	4
Standby	ea	1	--
Total	ea	4	4

Both methods yield the same result; a total of 4 digesters are needed. For cost estimating and layout purposes, the two new digesters are assumed to be the same size and configuration as the existing digesters. Support equipment, including heating, mixing, and gas handling facilities would also be required.

If the City wants to move towards Class A solids production in the future, the configuration of the digesters will likely be different to accommodate the Class A technology. The predesign for the new solids handling system should evaluate in detail the potential need for Class A solids and revise the digester layout accordingly.

3.11 Solids Dewatering

There are three existing 2-meter belt presses. Due to age and condition of the existing units, the ability to obtain drier solids, and odor control, the City is evaluating replacing the belt presses with centrifuges. The centrifuges will produce a drier sludge and therefore reduce hauling costs; however the power requirement of a centrifuge is much higher than that of a belt press. With either technology the dewatering process would be limited to approximately 7 hours per day to avoid the need for additional staffing outside of the normal work shifts. If one unit is out of service, redundancy could be provided by operating for a longer period until the unit is repaired.

The following table compares belt presses versus centrifuges for dewatering at Dry Creek.

Table 13 Dry Creek Dewatering Options

		Belt Press	Centrifuge
Governing Condition		Peak Month	Peak Month
Digested Sludge Flow	gpd	243,380	243,380
Dry Solids Loading	lb/day	43,990	43,990
Hours of Operation	hr	7	7
Sludge Flow to Dewatering	gpm	580	580
Number of Units Needed	ea	5	2
Loading per Duty Unit	gpm	120	300
Installed Horsepower per unit	HP	15	400
Cake Dry Solids	%	16	20

For layout and cost estimating purposes, the centrifuge option will be included. Assuming a centrifuge capacity of 250 – 350 gpm, which is similar to the units at Pleasant Grove, two duty centrifuges would be required.

3.12 Recycled Water Pumps

One additional recycled water pump would be located in the spare space at the existing recycled water pump station. Discussion of the recycled water pumps and their associated costs are discussed in the Recycled Water technical memorandum.

4 Pleasant Grove Expansion Recommendations

The Pleasant Grove treatment plant was completed in 2004 and reached full flow in February 2005. It was designed for an ADWF of 12 mgd, an average BOD loading of 16,000 lb/day, and an average TSS loading of 22,000 lb/day. The liquid treatment process includes screen and grit removal, influent pumping, secondary treatment/denitrification in oxidation ditches, secondary clarifiers, filtration, and disinfection with sodium hypochlorite. The solids treatment process is sludge dewatering with centrifuges.

The ADWF in 2005 was 6.6 mgd. However, the concentration of influent BOD to the plant is significantly higher than the concentration used during the plant design in 2000. The current BOD loading to the plant is 15,400 lb/day, or 96 percent of its nominal design BOD loading. The current TSS loading is 18,600 lb/day, or 84 percent of the design TSS loading.

The recommended expansion requirements are driven by three factors:

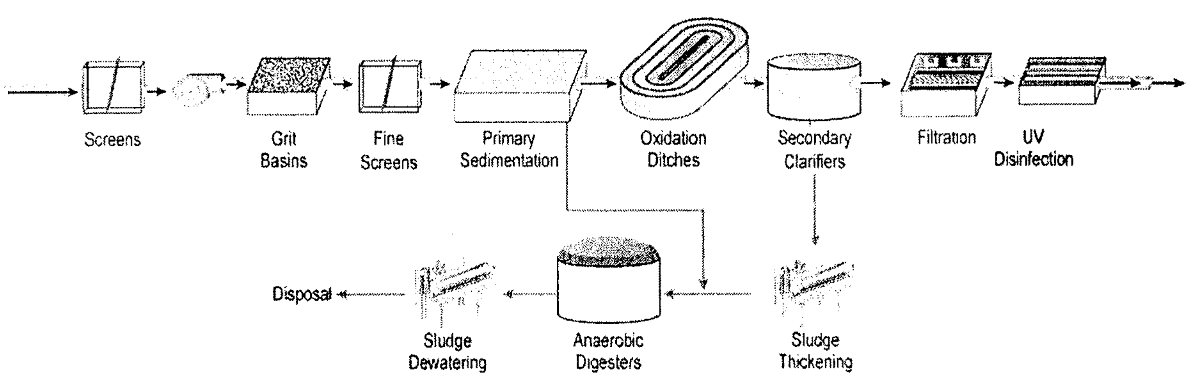
- Flow capacity to meet the anticipated ADWF of 24 mgd and peak hydraulic flow of 60 mgd (and space for an ultimate ADWF of 27 mgd)
- Organic treatment capacity to meet an anticipated BOD loading of 57,000 lb/day and a TSS loading of 68,100 lb/day
- Denitrification to meet an expected new NPDES limit of 10 mg/l-N

Two alternatives for expansion of the Pleasant Grove WWTP were initially evaluated. The first alternative expanded the existing treatment process train, i.e. used oxidation ditches without primary sedimentation. The second alternative adds primary sedimentation upstream of the oxidation ditches and adds solids thickening and anaerobic digesters to the solids treatment processes. After initial evaluation of the two alternatives and review by SPWA, the second alternative, adding primary sedimentation, was recommended for the following reasons:

- The total cost of Alternative 2 was approximately 6 percent less than the cost of Alternative 1. Meeting the buildout flows and loadings with the original process train would have required a total of 10 oxidation ditches and 8 centrifuges.
- Improving the sludge processing system by adding thickening and stabilization will eliminate the need for the WAS holding tanks, produce a more stabilized sludge, reduce the volume of sludge for dewatering, and reduce the odors associated with solids handling.
- Adding primary sedimentation will reduce the organic loading on the oxidation ditches and therefore reduce the amount of power needed for the aerators in the oxidation ditches
- The anaerobic digesters will produce methane which can be used in a co-generation process to produce electricity and reduce the plant's power consumption.
- The addition of fine screens upstream of the primary sedimentation process will remove fine plastics which currently accumulate in the oxidation ditches, the sludge, and in the recycled water process.

The rest of the TM focuses on the alternative of adding primary sedimentation and expanded solids handling to the Pleasant Grove process train. The resulting treatment processes are illustrated in Figure 3.

Figure 3 Pleasant Grove Treatment Schematic



The recommended facilities are summarized in Table 27 – Pleasant Grove Design Criteria and the proposed layout for the new facilities is shown on Figure 4. Both are included at the end of this TM.

The design basis for each process is discussed below.

4.1 Preliminary Treatment

4.1.1 Coarse Screens

The existing screening facility for the East Roseville trunk sewer includes 2 mechanically cleaned bar screens with ½ inch openings. There is an existing spare channel. The system expansion assumes the addition of one mechanically cleaned bar screen, similar to the two existing, in the existing spare channel.

4.1.2 Influent Pump Station

The existing influent pump station includes 2 low-range pumps rated at 9 mgd and 2 high-range pumps rated at 21 mgd. With the largest pump out of service the firm pumping capacity is 39 mgd. There are two spare spaces for additional pumps. The required firm capacity of the influent pump station at buildout is 60 mgd (PHWWF). This would require the addition of one more high range pump. With one large pump out of service, the resulting firm pumping capacity would be 60 mgd. For an ultimate PHWWF flow of 67.5 mgd, one additional low-range pump. The firm capacity would then be 69 mgd with the largest unit out of service.

There are two existing side stream pumps, each rated at 5.1 mgd. The projected side stream flow at peak day conditions is 4.3 mgd. The existing side stream pumps are adequate for buildout and no additional side stream pumps are needed.

4.1.3 Grit Chambers

The in-plant recycle streams are returned to the main plant flow upstream of the grit basins. These flows consist predominately of filter backwash, but also include spray water, filtrate from dewatering, and the new filtrate from the solids thickening process. With the addition of the in-plant recycle flows, the effective flow to the grit basins and downstream processes is as follows:

Table 14 Pleasant Grove Flow to Processes, Including Recycle Streams

		ADWF	Peak Month	PDWWF	PHWWF
At Buildout					
Influent Flow	mgd	24.0	33.6	48.0	60.0
Recycle Flows					
Spray water and washdown	mgd	0.2	0.3	0.3	0.3
Backwash Recycle	mgd	2.4	2.4	2.4	2.4
Dewatering Recycle	mgd	0.3	0.3	0.3	0.3
Thickening Recycle	mgd	0.6	0.7	0.7	0.7
Total Recycle Flows	mgd	3.5	3.7	3.7	3.7
Percent of Influent Flow	%	14.5%	11.0%	7.7%	6.2%
Total Flow with Recycle	mgd	**27.5**	**37.3**	**51.7**	**63.7**
At Ultimate					
Influent Flow	mgd	27	37.8	54	67.5
Total Recycle Flows	mgd	4.4	4.6	4.6	4.6
Total Flow with Recycle	mgd	**31.4**	**42.4**	**58.6**	**72.1**

There are two existing aerated grit chambers, each sized for an overflow rate of 42,900 gpd/sf at PHWWF. Using a similar overflow rate, two additional grit chambers would be needed for the buildout and ultimate flow as shown below.

Table 15 Pleasant Grove Grit Basin Sizing

		Buildout	Ultimate
Design Flow (PHWWF)	mgd	63.7	72.1
Design Overflow Rate	gpd/sf	42,900	42,900
Total Area Required	sqft	1,480	1,680
Existing Area	sqft	840	840
Additional Area Required	sqft	640	840
Additional Basins Needed	ea	2	2

4.1.4 Fine Screens

The ½ inch coarse screens allow plastics to pass through to downstream treatment processes. The plastics are neutrally buoyant and are not removed through the secondary process. Addition of a fine screen process is proposed to remove plastics.

The fine screens would be sized to pass the buildout PHWWF of 67.5 mgd with provisions to expand if needed for the ultimate flow. For layout and cost estimating purposes this evaluation assumes the addition of three 6 mm (1/4") band screens, each with a capacity of 22.5 mgd, built as part of the influent/flow splitting structure for the new primary sedimentation basins. The facility would also include odor control and a screenings washer/compactor to remove organics caught by the screens. One of the key issues to be resolved during predesign is an assessment of the hydraulic profile of the plant to determine whether there is adequate head available for the fine screens.

4.2 Primary Sedimentation

The following table summarizes the design criteria and required number of basins needed for a new primary sedimentation process.

Table 16 Pleasant Grove Primary Sedimentation Sizing

		ADWF	Peak Month	PDWWF	PHWWF
Flow Rate	mgd	27.5	37.3	51.7	63.7
Maximum Design Overflow Rate	gpd/sf	1,000	1,200	1,600	2,500
Area Required	sf	27,500	31,083	32,313	25,480
Rectangular Basins					
Length	ft	230	230	230	230
Width	ft	20	20	20	20
Area per Basin	sf	4,600	4,600	4,600	4,600
Number of Basins Needed	ea	6	7	7	6
Circular Basins					
Diameter	ft	125	125	125	125
Area per Basin	sf	12,266	12,266	12,266	12,266
Number of Basins Needed	ea	3	3	3	3

Two options are shown, rectangular primary sedimentation basins similar to those at Dry Creek, and circular basins. Fewer circular basins would be required and they are usually less costly to construct.

However, the space allocated for addition of a primary sedimentation process in the existing site plan is inadequate for three circular basins. Therefore, for layout and cost estimating purposes, the primary sedimentation process will include seven rectangular primary sedimentation basins, covered for odor control, and associated sludge pumping facilities. One additional primary sedimentation basin would be needed for the ultimate flow. The project will also include a biofilter for odor control of the primary sedimentation and the fine screens.

4.3 Secondary Treatment

There are three existing oxidation ditches, each 3.2 MG in volume, and four existing 125 ft diameter clarifiers. The design of the oxidation ditches for the system expansion is based on producing an effluent with a total nitrogen concentration of less than 10 mg/l-N at a mean cell residence time (MCRT) of 11 days during the summer and 13 days during the winter. The governing criteria for the oxidation ditch sizing is the peak month, wet weather condition with an MCRT of 13 days. Sizing of the clarifiers depends on meeting both a maximum hydraulic loading rate and solids loading rate. As discussed previously with Dry Creek, the sizing of the two components of the secondary treatment process also depends on striking a balance between the oxidation ditch volume and the clarifier area.

Two options for sizing the oxidation ditches and clarifiers were considered. The first option uses a mixed liquor suspended solids (MLSS) concentration of 2,500 mg/l in the summer and 3,000 mg/l in the winter. The second option uses a MLSS of 3,000 mg/in the summer and 3,500 mg/in the winter. The following table summarizes the process sizing for the two options at buildout flows and loadings.

Table 17 Pleasant Grove Secondary Treatment Sizing

		Option 1	Option 2
Oxidation Ditch Sizing			
Governing Condition		Peak Month	Peak Month
Total Secondary Sludge Produced	lb/day	36,730	36,730
MCRT Required	days	13	13
Required Solids for MCRT	lb	477,490	477,490
Design winter time MLSS	mg/l	3,000	3,500
Required Basin Volume for MCRT	MG	19.1	16.4
Volume Existing Basins	MG	9.6	9.6
Additional Volume Needed	MG	9.5	6.8
Additional Ditches Needed	ea	3	2
Clarifier Sizing			
Flow	mgd	37.3	37.3
Anticipated Total RAS Flow	mgd	24.00	30.00
MLSS	mg/l	3,000	3,500
Max Hydraulic Overflow Rate	gal/sqft-day	400	400
Maximum Solids Loading Rate	lb/sqft-day	20	20
Req'd Area for Hydraulic Loading	sqft	93,300	93,300
Req'd Area for Solids Loading	sqft	76,700	98,200
Area Existing Clarifiers	sqft	49,100	49,100
Additional Area Needed	sqft	44,200	49,100
Additional Clarifiers Needed	ea	4	4

The peak month loadings were the governing criteria for both the oxidation ditches and the clarifiers. The clarifier sizing is set by the hydraulic loading rate in Option 1 and by the solids loading rate in Option 2. Both options require the addition of four new clarifiers.

Based on an estimated peak oxygen demand of 90,000 lb/day, approximately 2,500 HP of installed aerators would be needed. For Option 1 with 3 additional oxidation ditches, this would be 420 HP per ditch. For Option 2 with only 2 additional ditches, the installed horsepower would be 500 HP per basin would be difficult to install with the ditch configuration and would detrimentally affect the denitrification performance of the anoxic zones. The recommendation is therefore to add 3 additional oxidation ditches for a total of 6, and 4 new clarifiers, for a total of 8.

The new oxidation ditches would be built to the south of the existing ditches. The new clarifiers, along with a RAS pump station, would be built to the south of the existing clarifiers.

For an ultimate flow of 27 mgd, one additional oxidation ditch and no additional clarifiers would be required.

4.4 Effluent Filtration

There are six existing continuous backwash filters, with a filter area of 500 square feet per filter. At a maximum filter surface loading of 5 gpm/sq ft (Department of Health Services guidelines for Title 22 reuse, this corresponds to a capacity of 21.6 mgd (approximately peak hour dry weather flow). The loading rate for the current design PDWWF of 30 mgd would be 6.9 gpm/sf

The filters must meet the DHS loading criteria of 5 gpm/sf for the peak hour of the ADWF, which is assumed to have a peaking factor of 1.5 over the ADWF. For the system expansion, a more conservative design criteria is recommended. Even though it is unlikely that recycled water would be used for irrigation during the peak wet weather day and thus would not have to meet the 5 gpm/sf loading rate recommended by DHS, the effluent must still meet a turbidity limit of less than 2 ntu. Maintaining the surface loading rate closer to 5 gpm/sf during PDWWF will help the plant meet the effluent turbidity of 2 ntu during wet weather events. The following table shows the design surface loading rates and the actual overflow rates resulting from adding additional filters similar in size and configuration to the existing filters.

Table 18 Pleasant Grove Filters Sizing Criteria

		ADWF	Peak Hour of ADWF	Peak Month	PDWWF
Buildout Flow Rate	mgd	27.5	41.25	37.3	51.7
Desired Design Overflow Rate	gpm/sf	5	5	5	5
Existing Filter Area	sf	3,819	5,729	5,181	7,181
Area per Additional Filter	sf	500	500	500	500
Overflow Rate with 6 addn'l filters [(1)]	sf	3.18	4.77	4.32	5.98
Overflow Rate with 8 addn'l filters[(1)]	gpm/sf	2.73	4.09	3.70	5.13
Ultimate Flow Rate	mgd	31.4	47.0	42.4	58.6
Overflow Rate with 9 addn'l filters[(1)]	gpm/sf	2.9	4.4	3.9	5.4

(1) Filter area assumes all units in service since filters are continuous backwash

Adding six additional filters would meet the DHW Title 22 criteria for all conditions at buildout except for peak day wet weather. The recommendation is therefore to add six new filters, but to provide space for additional units should future operating experience at the plant indicate that a lower overflow rate during wet weather is desirable. For an ultimate flow of 27 mgd, a total of 9 additional filters would be required, or 3 more than are added for buildout conditions. The original plant layout only provided space for four additional filters, so the filter layout will require additional analysis during predesign.

4.5 Disinfection

The Pleasant Grove WWTP currently disinfects with sodium hypochlorite. During the system expansion the disinfection method would be converted to UV disinfection. The proposed UV system will have five channels, with 4 UV banks per channel, similar in configuration to the UV system currently under design for Dry Creek. It would be built in the same location as the existing chlorine contact channels.

For an ultimate flow of 27 mgd, one additional UV channel would be required.

4.6 Recycled Water Pumps

The system expansion includes two additional recycled water pumps located in the spare space at the existing recycled water pump station. Discussion of the recycled water pumps and their associated costs are discussed in the Recycled Water technical memorandum

4.7 Solids Thickening

With the addition of primary sedimentation, the solids handling process will change to include solids thickening and anaerobic digestion prior to solids dewatering. Activated sludge would be wasted to the thickening process on a continuous, 24 hour per day process. This will eliminate the need for the two existing waste activated sludge (WAS) holding tanks, which are a significant source of odor from the plant.

There are several types of thickening processes which should be evaluated in predesign. The following table shows a comparison of gravity belt thickeners versus centrifuges.

Table 19 Pleasant Grove Sludge Thickening Options

		Gravity Belt Thickener	Centrifuge
Governing Condition		Peak Month	Peak Month
WAS Production	lb/day	36,730	36,730
WAS Production	mgd	0.57	0.57
Hours of Operation	hrs/day	24	24
WAS Flow to Thickeners	gpm	396	396
Maximum Unit Loading Rate	gpm/unit	400	400
Required Number of Thickeners			1
Duty	ea	1	1
Standby	ea	1	1
Total	ea	2	2

Centrifuges are higher in cost and have significantly higher power requirements than gravity belt thickeners, but can more easily operate unattended at night and require less odor control. For cost and layout purposes, centrifuges have been used as the thickening process, but further analysis, particularly of power requirements, should be done in predesign. Two thickening centrifuges (one duty and one standby) would be housed in a new solids thickening building.

No additional thickeners would be required for an ultimate flow of 27 mgd.

4.8 Anaerobic Digesters

Anaerobic digesters would be added for stabilization of the primary sludge and thickened WAS prior to dewatering. Addition of digesters will reduce the volume of sludge going to dewatering, reduce odor problems associated with storage and dewatering of non-stabilized sludge, and produce methane which can be used in co-generation equipment to produce electricity. The following table shows the sizing criteria used for new digesters at Pleasant Grove.

Table 20 Pleasant Grove Digester Sizing

		Buildout	Ultimate
Peak Month Solids Production	lb/day	86,640	97,465
Peak Month VSS Production	lb/day	69,220	77,864
VSS Design Criteria	lb VSS/cf/day	0.10	0.11
Total Volume Needed	MG	5.2	5.3
Digesters			
Diameter	ft	90	90
Side Water Depth	ft	28	28
Total Volume, each	MG	1.3	1.3
Total Digesters Needed		4	4
Detention Time	days	18.3	16.3

Four anaerobic digesters, each 90 ft in diameter and 28 ft deep, would be added. By operating at a slightly higher volatile solids loading rate, the same digesters would be sufficient for the ultimate loads. The digesters would be 90 ft diameter digesters for a total volume of 4.76 million gallons. A new digester control building would house support equipment including heating and mixing equipment for the digesters.

4.9 Solids Dewatering

There are three existing centrifuges for sludge dewatering. (the third centrifuge is currently in construction) For the buildout flow of 24 mgd, one additional centrifuge would be installed in a spare space in the existing solids handling building for a total of four (three duty and one standby). Assuming an operating period of 6 hours per day for dewatering, the loading per centrifuge would be 230 gpm.

No additional centrifuges would be needed for an ultimate ADWF flow of 27 mgd.

Table 21 Pleasant Grove Dewatering Criteria

		Buildout	Ultimate
Governing Condition		Peak Month	Peak Month
Digested Sludge Flow	gpd	290,500	326,800
Dry Solids Loading	lb/day	52,030	58,530
Hours of Operation	hr	7	7
Sludge Flow to Dewatering	gpm	690	780
Loading per Duty Unit	gpm	230	260
Number of Units Needed			
Duty	ea	3	3
Standby	ea	1	1
Total	ea	4	4
Installed Horsepower per unit	HP	400	400
Cake Dry Solids	%	20	20

4.10 Co-Generation Facilities

The options for using digester gas are cogeneration, hot water boilers or waste gas flaring. Cogeneration, also referred to as combined heat and power (CHP), uses the digester gas for generation of both power and heat and should be integrated into the new anaerobic digester system. The cogeneration system will be integrated into the digestion process by running on digester gas produced from the anaerobic digesters and producing heat for the digestion process.

The following table summarizes the three primary types of co-generation currently in use at wastewater treatment plants.

Table 22 Cogeneration Technology Comparison

	Internal Combustion Engine	Microturbine	Fuel Cell
Advantages	• High power efficiency • Can be overhauled on-site • Operates on low-pressure gas	• Fewer moving parts • Compact size • Low emission • No cooling required	• Can be installed outdoors • No combustion • Very low emission • Low noise
Disadvantages	• High maintenance costs • Requires cooling system • High levels of low frequency noise • Must be installed in a building for noise control	• High capital cost • Lower mechanical efficiency • Lower electrical efficiency • Major overhaul or replacement after 10 years	• Highest capital cost • Emerging technology without an extensive installation history • Major overhaul after 3-5 years
Installation	Indoors	Outdoors	Outdoors
Air Emissions	Relatively high	Low emissions	Very low emissions
Electrical Efficiency	22-40%	18-27%	43%

A significant amount of electricity could be generated from a co-generation facility at Pleasant Grove. The following table summarizes the estimated electrical power production based on average day loadings to the plant.

Table 23 Pleasant Grove Potential Co-generation Power Output

Item	Unit	Value
Volatile Solids, average	lb/day	59,030
Volatile Solids Destruction	%	50%
Digester Gas Generation	cu ft/lb VSS destroyed	15
Digester Gas Energy Content	BTU/cf	550
Total Energy Content	BTU/day	243,498,750
Total Energy Content	BTU/hr	10,145,781
Avg. Co-gen Electrical Efficiency	%	34%
Electrical Output	kw	1,010
Electrical Output	hp	1,350

The predesign should select the co-generation equipment based on capital cost, emissions requirements, available subsidies, and the cost of power. For cost estimating and layout purposes, two 500-kW lean burn internal combustion engines are assumed.

4.11 Standby Generators

For budgeting purposes, it is assumed that one additional standby power generator would be installed in the space allocated adjacent to Electrical Building No. 1.

5 Construction Phasing and Estimated Costs

The system expansion at each plant would be done in phases. For each plant, the first priority would be to add organic capacity since the current BOD loading is higher than anticipated during design of the plants. Increased organic treatment capacity is also needed in anticipation of changes in the NPDES permits to add a nitrate limit. Subsequent phases of construction would add both hydraulic and organic capacity to meet the projected needs at buildout. Phasing the construction will allow SPWA to evaluate the actual versus projected rate of growth in the service area and adjust the timing of expansions accordingly.

Construction costs are based on a December 2005 ENR of 8462 and include the following allowances:

- Sitework 10%
- Electrical and Controls 15%
- Contingency 30%
- Engineering and administration 25%

The construction costs are an order of magnitude estimate as defined by the American Association of Cost Engineers. The estimate is based on scale-up or down factors and is normally expected to be accurate within plus 50 percent or minus 30 percent.

The buildout timelines for both plants were developed and presented in the TM titled Wastewater Treatment Plant Projected Loadings and Buildout TM 4a. The timelines presented here have been modified to more realistically reflect future growth. This was accomplished by 1) staggering the start date of the UGAs to more accurately reflect the level of effort associated with approval of a UGA by local land use authorities and 2) extending the buildout dates to fiscal year 2030-31 to more accurately reflect historical absorption rates in the region.

The changes that were made to the timeline presented in TM 4a at the Dry Creek Wastewater Treatment Plant include:

1. Modifying the buildout date for the 2005 Roseville Service Area from 2020 to 2015
2. Extending the estimated buildout date for Placer Vineyards to fiscal year 2030

The changes that were made to the timeline presented in TM 4a at the Pleasant Grove WWTP include:

1. Extending the buildout date for Placer Ranch to fiscal year 2030
2. Pushing the start date for Curry Creek to fiscal year 2014 and extending the buildout date to fiscal year 2034
3. Pushing back the start date for Creekview and Sierra Vista to fiscal year 2010
4. Extending the buildout date for Creekview, Sierra Vista, Regional University, and Orchard Creek to fiscal year 2030

5.1 Dry Creek Expansion

Figure 5 shows the projected timeline for buildout for construction phasing of the Dry Creek plant. The y-axis shows the influent flows and the corresponding BOD loading. The expansion of the Dry Creek plant is proposed to occur in two phases. The first priority at Dry Creek is to add organic treatment capacity in response to the increased BOD concentrations in the influent. The Phase 1 construction would increase the BOD capacity from the current 24,000 lb/day up to 34,500 lb/day, which corresponds to an influent flow of 15 mgd, and would be on line in FY 2010/11. Phase 1 would include a new influent pump station, fine screens, new aeration basins and clarifiers north of the existing aeration basins, an additional digester, and new centrifuges for dewatering. The second phase of construction would be completed in FY 2016/17 and would increase the plant capacity up to the buildout flow estimate of 21 mgd. The second phase of construction would include new grit and primary sedimentation basins, the replacement of the older aeration basins and clarifiers, and expansion of other facilities for the increased hydraulic load.

Figure 5 Dry Creek Construction Phasing

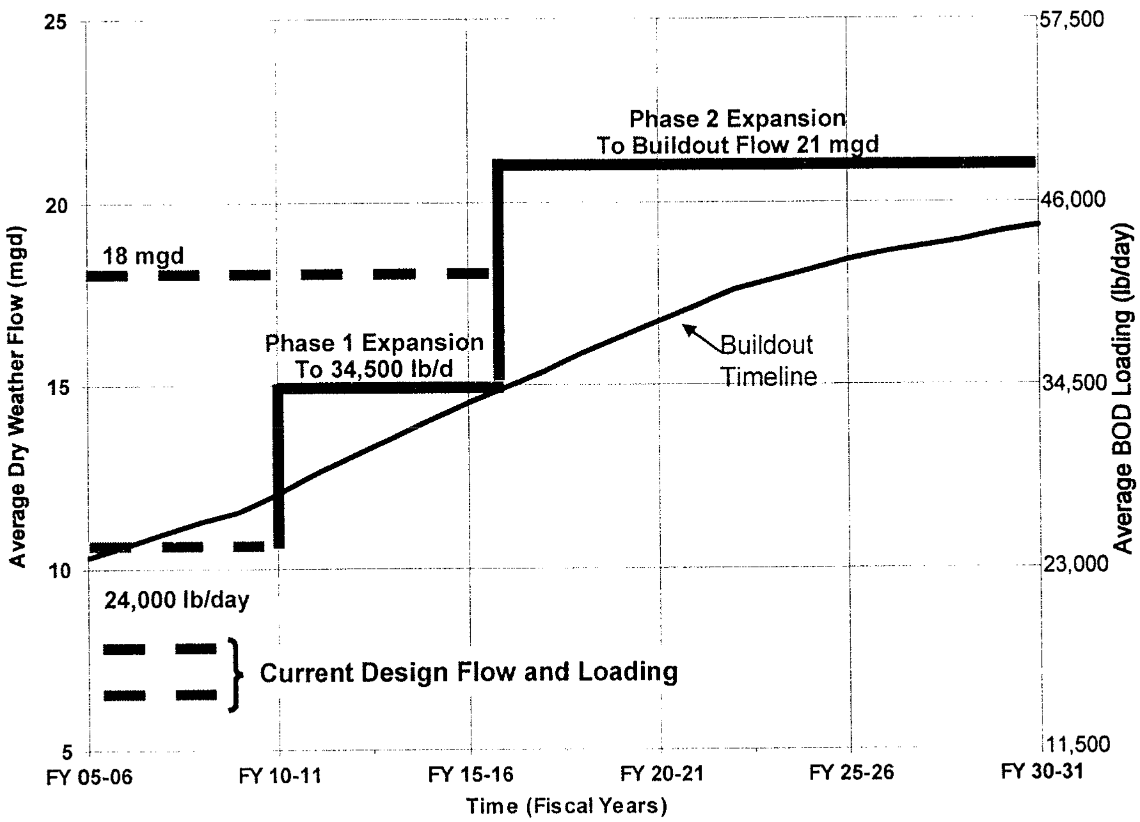


Table 24 summarizes the facilities and costs associated with each construction phase. A more detailed estimate is included in the Attachments.

Table 24 Dry Creek Construction Phasing and Estimated Costs

Construction Phase	Phase 1 Construction				Phase 2 Construction			
Year on-line	FY 2010/11				FY 2016/17			
ADWF Capacity	15 mgd				21 mgd			
Process	Number	Construction Cost	Eng & Admin	Total Cost	Number	Construction Cost	Eng & Admin	Total Cost
Influent Screens	1	$80,000	$20,000	$100,000				
Influent Pumps	4	$1,938,000	$430,000	$2,368,000	1	$342,000	$143,000	$485,000
Fine Screens	2	$500,000	$110,000	$610,000	1	$89,000	$37,000	$126,000
Odor Control	1	$120,000	$30,000	$150,000	1	$120,000	$30,000	$150,000
Grit Basins					1	$330,000	$83,000	$413,000
Primary Sedimentation					2	$4,890,000	$1,223,000	$6,113,000
Aeration Basins	6	$17,610,000	$4,400,000	$22,010,000	4	$11,740,000	$2,940,000	$14,680,000
Blower	1	$330,000	$83,000	$413,000				
Rehab Exst Anoxic Zones	1	$330,000	$83,000	$413,000				
Secondary Clarifiers	4	$11,740,000	$2,940,000	$14,680,000	2	$5,870,000	$1,470,000	$7,340,000
RAS/WAS Pump Station	1	$980,000	$245,000	$1,225,000	1	$980,000	$245,000	$1,225,000
Tertiary Filtration					2	$1,660,000	$415,000	$2,075,000
Waste Backwash Pumps					1	$110,000	$28,000	$138,000
UV Disinfection					1	$2,330,000	$583,000	$2,913,000
Anaerobic Digesters	1	$2,940,000	$730,000	$3,670,000	1	$2,940,000	$730,000	$3,670,000
Dewatering Centrifuges	2	$1,470,000	$368,000	$1,838,000				
Cooling Units					2	$650,000	$163,000	$813,000
Totals		**$38,000,000**	**$9,400,000**	**$47,400,000**		**$32,100,000**	**$8,100,000**	**$40,200,000**

5.2 Pleasant Grove Expansion

Figure 6 shows the projected timeline for buildout for construction phasing of the Pleasant Grove plant The y-axis shows the influent flows and the corresponding BOD loading. The expansion of the Pleasant Grove plant is proposed to occur in two phases. The first priority at Pleasant Grove is to add organic treatment capacity by constructing primary sedimentation, expand the aeration capacity, and add solids thickening and stabilization. The Phase 1 construction would increase the BOD capacity from the current 16,000 lb/day up to 36,000 lb/day and expand the hydraulic capacity to 15 mgd. The Phase 1 construction should take place quickly since the plant is already near its organic treatment capacity. It is expected that predesign, environmental permitting, final design, and construction will take approximately 4 years. The Phase 1 construction could therefore be on line in FY 2010/11. The second phase of construction would be completed in FY 2016/17 and would increase the plant capacity up to the buildout flow estimate of 24 mgd. The second phase of construction would include expansion of all of the processes to meet the buildout flows and loadings.

Figure 6 Pleasant Grove Construction Phasing

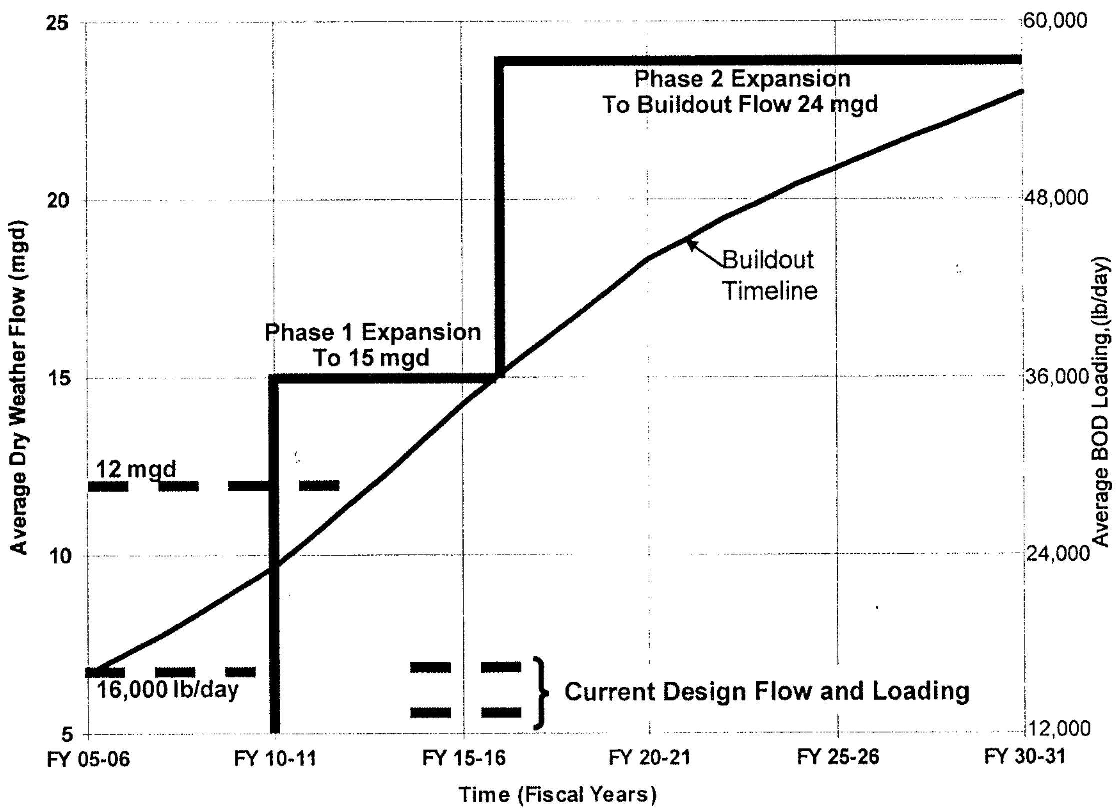


Table 25 summarizes the facilities and costs associated with each construction phase. A more detailed estimate is included in the Attachments.

Table 25 Pleasant Grove Construction Phasing and Estimated Costs

Construction Phase Year on-line ADWF Capacity	Phase 1 Construction FY 2010/11 15 mgd				Phase 2 Construction FY 2016/17 24 mgd			
Process	Number	Construction Cost	Engineering & Admin	Total Cost	Number	Construction Cost	Engineering & Admin	Total Cost
Influent Screens					1	$80,000	$20,000	$100,000
Influent Pumps	1	$140,000	$35,000	$175,000				
Grit Basins	1	$325,000	$82,000	$407,000	1	$325,000	$82,000	$407,000
Fine Screens	2	$443,000	$111,000	$554,000	1	$148,000	$37,000	$185,000
Primary Sedimentation	4	$9,783,000	$2,446,000	$12,229,000	3	$7,337,000	$1,834,000	$9,171,000
Odor Control	1	$120,000	$30,000	$150,000	1	$120,000	$30,000	$150,000
Oxidation Ditches	1	$8,070,000	$2,018,000	$10,088,000	2	$16,140,000	$4,035,000	$20,175,000
Secondary Clarifiers	1	$2,935,000	$734,000	$3,669,000	3	$8,805,000	$2,201,000	$11,006,000
RAS/WAS Pump Station	1	$490,000	$123,000	$613,000	modify	$490,000	$123,000	$613,000
Tertiary Filtration	2	$1,660,000	$420,000	$2,080,000	4	$3,330,000	$830,000	$4,160,000
UV Disinfection	3	$6,984,000	$1,746,000	$8,730,000	2	$4,656,000	$1,164,000	$5,820,000
Centrifuge Thickeners	2	$1,470,000	$368,000	$1,838,000				
Building	1	$1,710,000	$428,000	$2,138,000				
Anaerobic Digesters	3	$8,805,000	$2,201,000	$11,006,000	1	$2,935,000	$734,000	$3,669,000
Building	1	$1,283,000	$321,000	$1,604,000	modify	$428,000	$107,000	$535,000
Centrifuge Dewatering					1	$730,000	$183,000	$913,000
Co-generation	1	$2,445,000	$612,000	$3,057,000	1	$2,445,000	$612,000	$3,057,000
Standby Generator					1	$440,000	$110,000	$550,000
Total		**$46,700,000**	**$11,700,000**	**$58,300,000**		**$48,300,000**	**$12,100,000**	**$60,400,000**

Table 26 Dry Creek Design Criteria for Buildout Flows and Loadings

Process	Units	Existing	Buildout
Influent Loadings			
Average Dry Weather Flow (ADWF)	mgd	18	21
Diurnal Peak Hour Flow	mgd	27	31.5
Peak Day Wet Weather Flow (PDWWF)	mgd	45	52.5
Peak Hour Wet Weather Flow	mgd	54	63
Average BOD Concentration	mg/l	160	275
Average BOD Load	lb/day	24,000	48,100
Peak Month BOD Peaking Factor	–		1.2
Average TSS Concentration	mg/l	240	310
Average TSS Load	lb/day	36,000	54,300
Peak Month TSS Peaking Factor	–		1.2
Average NH_3-N Concentration	mg/l		28
Average NH_3-N Load	lb/day		4,900
Screening			
Climber Screens	ea	2	3
Channel Velocity at PHWWF	fps	1.6	1.9
Bypass Manual Screen	ea	1	1
Influent Pump Station (excludes North Roseville force main)			
Number	ea	7	5
Capacity, ea	gpm	3000	8000
Horsepower	HP	75	175
Fine Screens			
Band Screens, Size	in		1/4
Number	ea	–	3
Capacity, ea	mgd		15
Grit Removal			
Type		Aerated	Aerated
Number of Basins	ea	2	3
Detention time at ADWF	min	7.8	9.2
Primary Sedimentation			
Number of Basins	ea	4	6
Size			
Length	ft	225	225
Width	ft	20	20
Side Water Depth (SWD)	ft	10	10
Overflow Rate at			
ADWF	gpd/sf	1,000	820
PDWWF	gpd/sf	2,500	2,030
Odor Control			
Type		Biofilter	

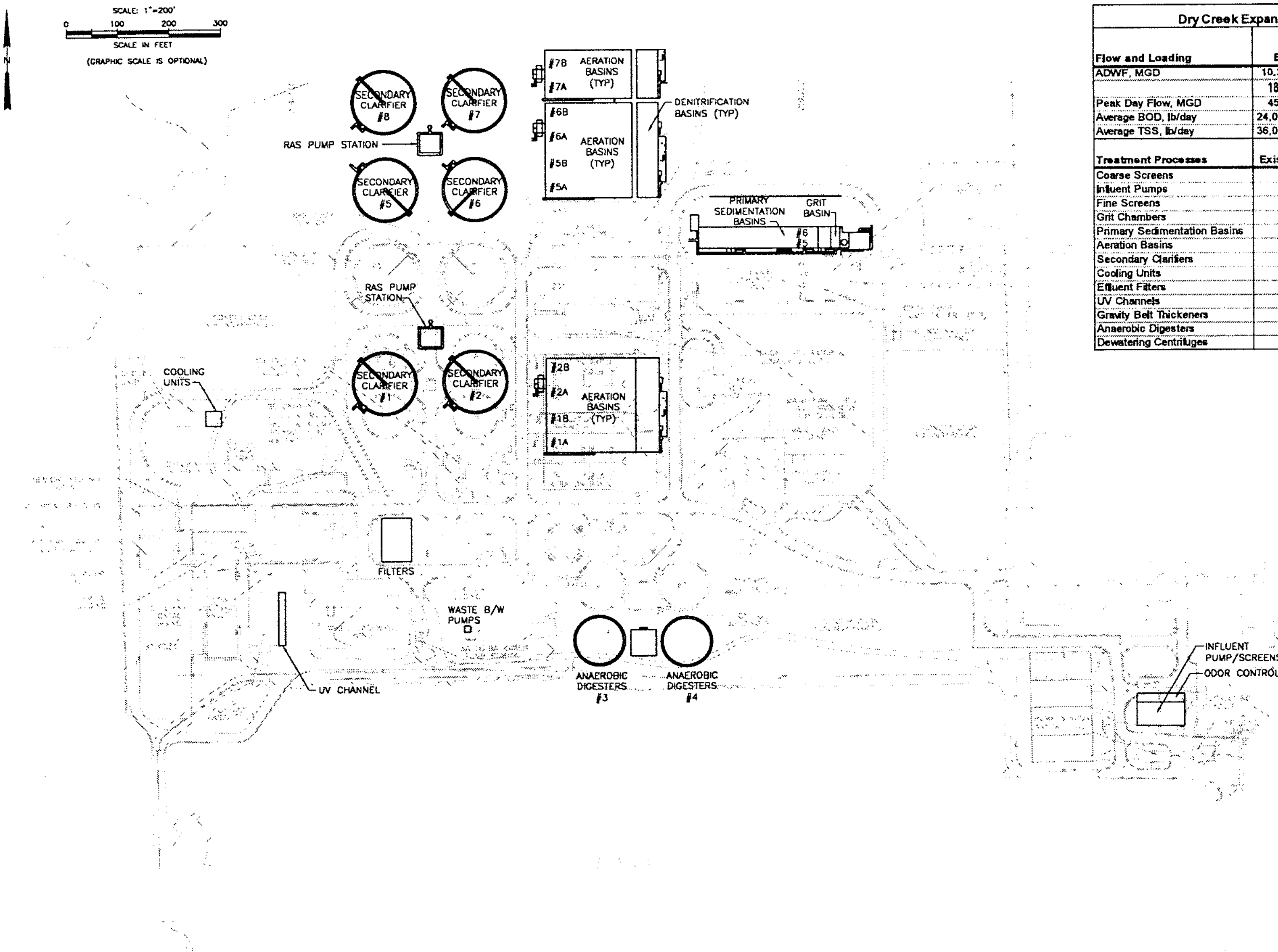
Process	Units	Existing	Buildout
Denitrification and Aeration Basins			
Number of Basins	ea	8 (Note [a])	14
Size of New Basins			
Anoxic Basins			
Length	ft		60
Width	ft		45
Side Water Depth (SWD)	ft		15
Aeration Basins			
Length	ft		150
Width	ft		45
Side Water Depth (SWD)	ft		15
Target MCRT at ADWF	days		11
Design MLSS at ADWF	mg/l		3,000
Peak Air Demand	scfm		35,300
Number of Blowers	ea	4	5
Secondary Clarifiers			
Number of Clarifiers	ea	6 (Note [b])	8
Diameter	ft		125
SWD	ft		16
Overflow Rate at			
ADWF	gpd/sf		230
PDWWF	gpd/sf		570
Total RAS Flow at ADWF	mgd		14
Total WAS Produced at ADWF	lb/day		27,500
Number of RAS Pump Stations	ea	2	3
Cooling Units			
Number	ea	4	6
Filtration			
Number of Filters	ea	3	5
Cells per Filter	ea	4	4
Area per Cell	sf	347	347
Total Filter Area	sf	4,164	6,940
Filter Loading (one cell out of service)	gpm/sf		
ADWF		3.2	2.4
PDWWF		7.9	5.9
Waste Backwash Pumps	ea	2	3
Disinfection			
Type		Ultraviolet Disinfection	
Design Dose	mj/cm sq	100	100
Number of Channels	ea	5	6
Number of UV Banks per channel	ea	4	4
Recycled Water Pumps			
Number	ea	3	4

Process	Units	Existing	Buildout
Sludge Thickening			
Type		Gravity Belt Thickeners	
Number of Units, Duty/Standby	ea	1/1	1/1
Size	meters		2
Hours of Operation	hrs		24
WAS Production at ADWF	mgd		0.46
Hydraulic Loading	gpm/meter		160
Thickened Sludge Concentration	%		5
Sludge Digestion			
Type		Anaerobic Digesters	
Number of Digesters	ea	2	4
Diameter	ft	90	90
SWD	ft	25	25
Total Volume	MG	2.4	4.8
Volatile Solids Loading Rate	lb/cf/dy	0.083	0.10
Detention @ ADWF	days	24	23
Sludge Control Buildings	ea	1	2
Sludge Dewatering			
Type		Centrifuges	
Number of Units	ea	N/A	2
Size	gpm		250-350
Hours of Operation	hrs		7
Digested Sludge Production at ADWF	gpd		205,000
Loading per centrifuge	gpm		245
Cake Dry Solids	%		20

Notes

[a] The four oldest aeration basins will be replaced with four new basins matching the size and configuration of the new basins

[b] The four oldest clarifiers will be replaced with two new 125 ft diameter clarifiers and a new RAS pump station



Dry Creek Expansion Recommendations

Flow and Loading	Existing	Phase 1 Expansion	Phase 2 Expansion	
ADWF, MGD	10.3 (Current)	15	21	
	18 (Design)			
Peak Day Flow, MGD	45 (Design)	37.5	52.5	
Average BOD, lb/day	24,000 (Design)	34,500	48,100	
Average TSS, lb/day	36,000 (Design)	33,800	54,300	
Treatment Processes	**Existing Units**	**Phase 1 Expansion**	**Phase 2 Expansion**	**Total Units at Buildout**
Coarse Screens	2	1	0	3
Influent Pumps	7	4	1	5
Fine Screens	N/A	2	1	3
Grit Chambers	2	0	1	3
Primary Sedimentation Basins	4	0	2	6
Aeration Basins	4	6	4	14
Secondary Clarifiers	2	4	2	8
Cooling Units	4	0	2	6
Effluent Filters	3	0	2	5
UV Channels	5	0	1	6
Gravity Belt Thickeners	2	0	0	2
Anaerobic Digesters	2	1	1	4
Dewatering Centrifuges	N/A	2	0	2

0" 1"
VERIFY SCALES
BAR IS ONE INCH LONG ON FULL SIZE DRAWING.
IF NOT ONE INCH LONG ON THIS DRAWING, ADJUST SCALES ACCORDINGLY

RMC Water & Environment

REV	DATE	BY	APVD	DESCRIPTION

DESIGNED
DRAWN
CHECKED

SUBMITTED: RMC PROJ MNGR
APPROVED: RMC MNGR

SPWA WASTEWATER SYSTEMS EVALUATION

DRY CREEK WWTP EXPANSION
FIGURE 2

DWG NO
SHEET NO OF
PROJ NO 0091-004
DATE JANUARY 2006

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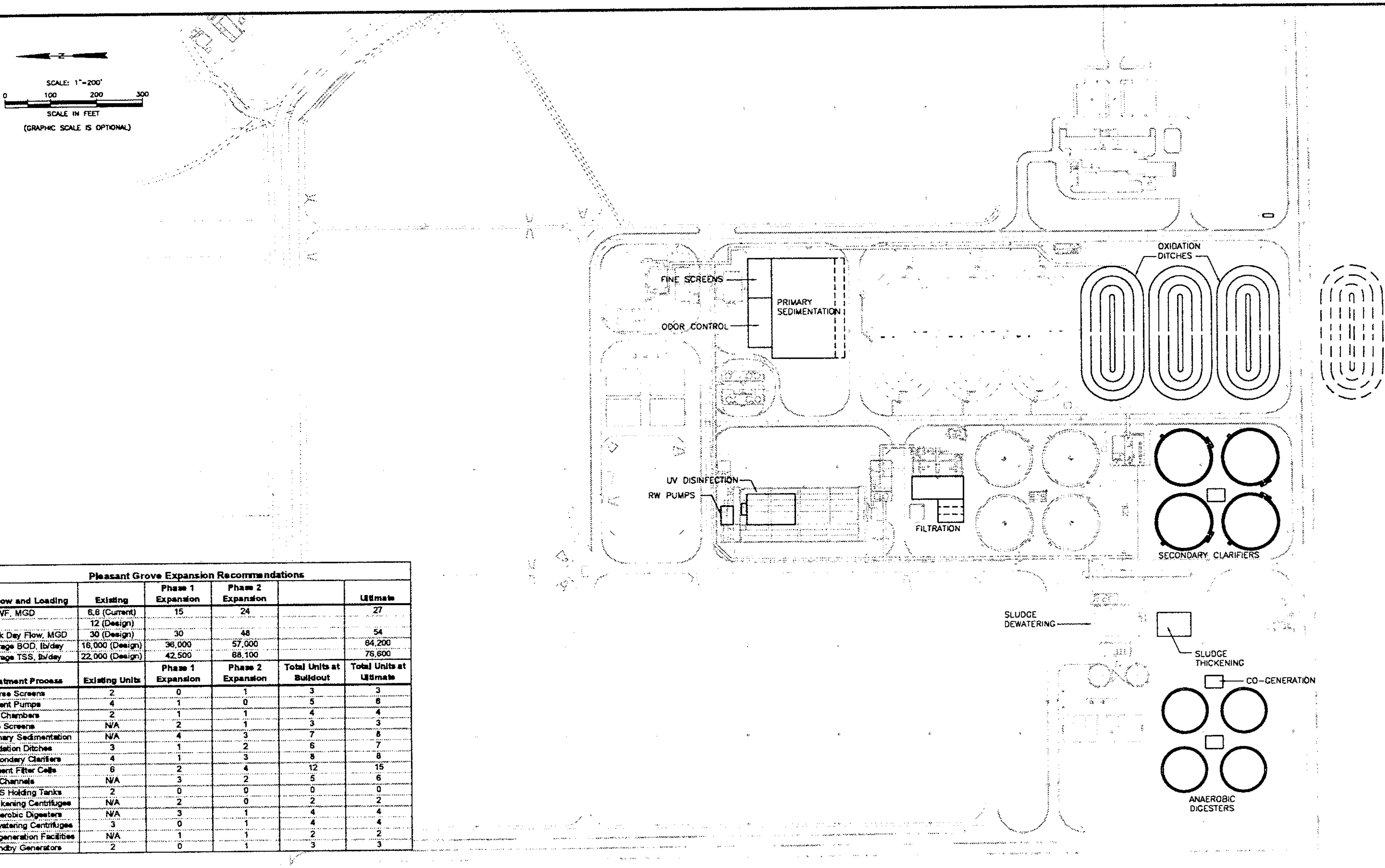
Table 27 Pleasant Grove Design Criteria for Expansion

Process	Units	Existing	Buildout	Ultimate
Influent Loadings				
Average Dry Weather Flow (ADWF)	mgd	12	24	27
Diurnal Peak Hour Flow	mgd	18	36	40.5
Peak Day Wet Weather Flow (PDWWF)	mgd	30	48	54
Peak Hour Wet Weather Flow (PHWWF)	mgd	36	60	67.5
Average BOD Concentration	mg/l	160	285	285
Average BOD Load	lb/day	16,000	57,000	64,000
Peak Month BOD Peaking Factor	–		1.2	1.2
Average TSS Concentration	mg/l	220	340	340
Average TSS Load	lb/day	22,000	68,000	77,000
Peak Month TSS Peaking Factor	–		1.2	1.2
Average NH_3-N Concentration	mg/l		28	28
Average NH_3-N Load	lb/day		5,600	6,300
Preliminary Treatment				
Screens	ea	2	3	3
Channel Velocity at PHWWF	fps	0.86	0.95	1.1
Raw Sewage Pumps	ea	4	5	6
Low Range, 9 mgd	ea	2	2	3
High Range, 21 mgd	ea	2	3	3
Aerated Grit Chambers	ea	2	4	4
Width	ft	14	14	14
Length	ft	30	30	30
Overflow Rate at ADWF	gpd/sf	14,300	14,300	16,100
Fine Screens				
Type		N/A	Band Screens	
Size			1/4	1/4
Number			3	4
Primary Sedimentation				
Number of Basins	ea	N/A	7	8
Size of New Basins				
Length	ft		225	225
Width	ft		20	20
Side Water Depth (SWD)	ft		10	10
Overflow Rate at				
ADWF	gpd/sf		870	850
PDWWF	gpd/sf		1,620	1,590
Odor Control				
Type			Biofilter	
Secondary Treatment				
Type			Oxidation Ditches	
Number	ea	3	6	7
Volume, each	MG	3.2	3.2	3.2
Target MCRT @ ADWF	days		11	11
Design MLSS at ADWF	mg/l		3,000	3,000

Process	Units	Existing	Buildout	Ultimate
Secondary Clarifiers				
Number of Clarifiers	ea	4	8	8
Diameter	ft	125	125	125
SWD	ft	14	14	14
Overflow Rate at				
ADWF	gpd/sf	326	290	320
PDWWF	gpd/sf	610	530	600
Total RAS Flow at ADWF	mgd		16	16
Total WAS Produced at ADWF	lb/day		32,000	36,000
Number of RAS Pump Stations	ea	1	2	2
Filtration				
Number of Filters	ea	6	12	15
Cells per Filter	ea	10	10	10
Area per Cell	sf	50	50	50
Total Filter Area	sf	3,000	6,000	7,500
Filter Loading (all cells in service)				
ADWF	gpm/sf	3.3	3.3	2.9
PDWWF	gpm/sf	7.7	6.0	5.4
Disinfection				
Type		Chlorine	Ultraviolet Disinfection	
Design Dose	mj/cm sq	N/A	100	100
Number of Channels	ea		5	6
Number of UV Banks per channel	ea		4	4
Recycled Water Pumps				
Number	ea	2	4	4
Sludge Holding Tanks				
Number of Units	ea	2	N/A	N/A
Volume	MG	0.34		
Detention Time	days	2		
Sludge Thickening				
Type		N/A	Centrifuges	
Number of Units, Duty/Standby	ea		1/1	1/1
Size	gpm		350-450	350-450
Hours of Operation	hrs		24	24
WAS Production at ADWF	mgd		0.52	0.54
Loading per centrifuge at ADWF	gpm		360	375
Thickened Sludge Concentration	%		5	5

Process	Units	Existing	Buildout	Ultimate
Sludge Digestion				
Type		N/A	Anaerobic Digesters	
Number of Digesters	ea		4	4
Diameter	ft		90	90
SWD	ft		28	28
Total Volume	MG		5.2	5.2
Volatile Solids Loading Rate	lb/cf/dy		0.10	0.11
Detention @ ADWF	days		22	19
Sludge Control Buildings	ea		1	1
Sludge Dewatering				
Type			Centrifuges	
Number of Units, Duty/Standby	ea	2/1	3/1	3/1
Size	gpm		250-350	250-350
Hours of Operation	hrs		7	7
Digested Sludge Production at ADWF	gpd		245,500	276,200
Loading per centrifuge at ADWF	gpm		195	220
Cake Dry Solids	%		20	20
Co-Generation				
Type			TBD	
Size	kW		1000	1000
Standby Generators				
Number		2	3	3



Pleasant Grove Expansion Recommendations					
Flow and Loading	Existing	Phase 1 Expansion	Phase 2 Expansion		Ultimate
ADWF, MGD	6.8 (Current)	15	24		27
	12 (Design)				
Peak Day Flow, MGD	30 (Design)	30	48		54
Average BOD, lb/day	16,000 (Design)	36,000	57,000		84,200
Average TSS, lb/day	22,000 (Design)	42,500	68,100		76,600
Treatment Process	Existing Units	Phase 1 Expansion	Phase 2 Expansion	Total Units at Buildout	Total Units at Ultimate
Coarse Screens	2	0	1	3	3
Influent Pumps	4	1	0	5	6
Grit Chambers	2	1	1	4	4
Fine Screens	N/A	2	1	3	3
Primary Sedimentation	N/A	4	3	7	8
Oxidation Ditches	3	1	2	6	7
Secondary Clarifiers	4	1	3	8	8
Effluent Filter Cells	6	2	4	12	15
UV Channels	N/A	3	2	5	6
WAS Holding Tanks	2	0	0	0	0
Thickening Centrifuges	N/A	2	0	2	2
Anaerobic Digesters	N/A	3	1	4	4
Dewatering Centrifuges	3	0	1	4	4
Co-generation Facilities	N/A	1	1	2	2
Standby Generators	2	0	1	3	3

NOTE:

——— FACILITIES AT BUILDOUT

— — — FACILITIES AT ULTIMATE

0" 1" — VERIFY SCALES — BAR IS ONE INCH LONG ON FULL SIZE DRAWING. IF NOT ONE INCH LONG ON THIS DRAWING, ADJUST SCALES ACCORDINGLY

RMC Water & Environment

REV	DATE	BY	APVD	DESCRIPTION

DESIGNED
DRAWN
CHECKED

SUBMITTED: RMC PROJ ENGR C
APPROVED: RMC ENGR C

SPWA WASTEWATER SYSTEMS EVALUATION

PLEASANT GROVE WWTP
FIGURE 4

DWG NO
SHEET NO OF
PROJ NO 0091-004
DATE JANUARY 2006

Last Saved By: 3-07-06 10:20am

ATTACHMENTS
COST ESTIMATES

Dry Creek Cost Estimate

Process	Quantity	Unit Price	ENR	Current ENR	Adjust Unit Cost	Total Process Cost	Sitework,Electrical & I&C @ 25%	Conting @ 30%	Total Construction Cost	E&A @ 25%	Total Cost
Coarse Screens	1	$50,000	8435	8462	$50,160	$50,000	$63,000	$19,000	$80,000	$20,000	$100,000
Influent Pump Station	1	$1,400,000	8435	8462	$1,404,481	$1,404,000	$1,755,000	$527,000	$2,280,000	$570,000	$2,850,000
Fine Screens	3	$120,000	8462	8462	$120,000	$360,000	$450,000	$135,000	$590,000	$148,000	$738,000
Odor Control	1	$150,000	8462	8462	$150,000	$150,000	$188,000	$56,000	$240,000	$60,000	$300,000
Grit Basins	1	$200,000	8435	8462	$200,640	$201,000	$251,000	$75,000	$330,000	$83,000	$413,000
Primary Sedimentation	2	$1,500,000	8435	8462	$1,504,801	$3,010,000	$3,763,000	$1,129,000	$4,890,000	$1,223,000	$6,113,000
Aeration Basins	10	$1,800,000	8435	8462	$1,805,762	$18,058,000	$22,573,000	$6,772,000	$29,350,000	$7,338,000	$36,688,000
Blower	1	$200,000	8435	8462	$200,640	$201,000	$251,000	$75,000	$330,000	$83,000	$413,000
Rehab Exst Anoxic Zones	1	$200,000	8462	8462	$200,000	$200,000	$250,000	$75,000	$330,000	$83,000	$413,000
Secondary Clarifiers	6	$1,800,000	8435	8462	$1,805,762	$10,835,000	$13,544,000	$4,063,000	$17,610,000	$4,403,000	$22,013,000
RAS/WAS Pump Station	2	$600,000	8435	8462	$601,921	$1,204,000	$1,505,000	$452,000	$1,960,000	$490,000	$2,450,000
Tertiary Filtration	2	$500,000	8271	8462	$511,546	$1,023,000	$1,279,000	$384,000	$1,660,000	$415,000	$2,075,000
Waste Backwash Pumps	1	$70,000	8462	8462	$70,000	$70,000	$88,000	$26,000	$110,000	$28,000	$138,000
UV Disinfection	1	$1,400,000	8271	8462	$1,432,330	$1,432,000	$1,790,000	$537,000	$2,330,000	$583,000	$2,913,000
Anaerobic Digesters	2	$1,800,000	8435	8462	$1,805,762	$3,612,000	$4,515,000	$1,355,000	$5,870,000	$1,468,000	$7,338,000
Centrifuges	2	$450,000	8435	8462	$451,440	$903,000	$1,129,000	$339,000	$1,470,000	$368,000	$1,838,000
Cooling Units	2	$200,000	8462	8462	$200,000	$400,000	$500,000	$150,000	$650,000	$163,000	$813,000
Total									**$70,000,000**	**$17,506,000**	**$87,506,000**

Pleasant Grove Cost Estimate

Process	Quantity	Unit Price	ENR	Current ENR	Adjust Unit Cost	Total Process Cost	Sitework,Electrical & I&C @ 25%	Conting @ 30%	Total Construction Cost	E&A @ 25%	Total Cost
Influent Screens	1	$50,000	8435	8462	$50,160	$50,000	$63,000	$19,000	$80,000	$20,000	$100,000
Influent Pumps	1	$85,000	8435	8462	$85,272	$85,000	$106,000	$32,000	$140,000	$35,000	$175,000
Grit Basins	2	$200,000	8462	8462	$200,000	$400,000	$500,000	$150,000	$650,000	$163,000	$813,000
Fine Screens	3	$120,000	8462	8462	$120,000	$360,000	$450,000	$135,000	$590,000	$148,000	$738,000
Primary Sedimentation	7	$1,500,000	8435	8462	$1,504,801	$10,534,000	$13,168,000	$3,950,000	$17,120,000	$4,280,000	$21,400,000
Odor Control	1	$150,000	8435	8462	$150,480	$150,000	$188,000	$56,000	$240,000	$60,000	$300,000
Oxidation Ditches	3	$4,600,000	7840	8462	$4,964,949	$14,895,000	$18,619,000	$5,586,000	$24,210,000	$6,053,000	$30,263,000
Secondary Clarifiers	4	$1,800,000	8435	8462	$1,805,762	$7,223,000	$9,029,000	$2,709,000	$11,740,000	$2,935,000	$14,675,000
RAS/WAS Pump Station	1	$600,000	8435	8462	$601,921	$602,000	$753,000	$226,000	$980,000	$245,000	$1,225,000
Tertiary Filtration	6	$500,000	8271	8462	$511,546	$3,069,000	$3,836,000	$1,151,000	$4,990,000	$1,248,000	$6,238,000
UV Disinfection	5	$1,400,000	8271	8462	$1,432,330	$7,162,000	$8,953,000	$2,686,000	$11,640,000	$2,910,000	$14,550,000
Centrifuge Thickeners	2	$450,000	8435	8462	$451,440	$903,000	$1,129,000	$339,000	$1,470,000	$368,000	$1,838,000
Building	3500	$300	8462	8462	$300	$1,050,000	$1,313,000	$394,000	$1,710,000	$428,000	$2,138,000
Anaerobic Digesters	4	$1,800,000	8435	8462	$1,805,762	$7,223,000	$9,029,000	$2,709,000	$11,740,000	$2,935,000	$14,675,000
Building	3500	$300	8462	8462	$300	$1,050,000	$1,313,000	$394,000	$1,710,000	$428,000	$2,138,000
Centrifuge Dewatering	1	$450,000	8435	8462	$451,440	$451,000	$564,000	$169,000	$730,000	$183,000	$913,000
Co-gen units	1000	$3,000	8435	8462	$3,010	$3,010,000	$3,763,000	$1,129,000	$4,890,000	$1,223,000	$6,113,000
Standby Generator	1	$270,000	8462	8462	$270,000	$270,000	$338,000	$101,000	$440,000	$110,000	$550,000
Total									**$94,990,000**	**$23,752,000**	**$118,742,000**

Technical Memorandum

South Placer Regional Wastewater and Recycled Water Systems Evaluation Project
City of Roseville Sanitary Sewer Model Development Project

Subject: Methodology for Adjusting Land Use for Parcels with Approved or Near-Certain Zoning or Development Changes ("Rezone Parcels") (TM No. 9b)

Prepared for: Art O'Brien – City of Roseville
Kenneth Glotzbach – City of Roseville

Prepared by: Gisa Ju, RMC

Reviewed by: Dave Richardson, RMC
Pete Bellows/Chris Peters, BC

Date: Revised October 24, 2005

Reference: 0091-3.05; 0091-4.01

The purpose of the memorandum is to recommend the methodology to be used to adjust the land use database being used to compute flows for the Roseville and SPWA sewer models to account for parcels identified since June 2004 as having zoning or development changes. These "rezones" will be incorporated into the future modeling scenarios for the Roseville and SPWA models.

The City of Roseville Planning Department provided information on 19 areas with changes in zoning, land use, or development intensity that have been approved since June 2004 or are considered likely to be approved in the near future. The list and description of these areas are attached in the document titled "Zoning, Land Use changes and development intensifications" provided by the City on September 28, 2005. The City also provided a GIS shape file with each of these areas shown as a discrete polygon.

Based on the GIS rezone area polygons, RMC determined the specific parcels included in each of the rezone areas. It should be noted that in some cases, the areas included portions of parcels rather than entire parcels. In these cases, the parcel was considered to be included in the rezone area if the majority of the parcel area fell within the rezone area polygon.

The attached table lists the rezone parcels, along with their current and originally projected land use and proposed rezone land use. Most of the rezones are conversions from non-residential to residential uses. The comments column in the table indicates the proposed method of calculating the land use information or flow for each parcel based on the rezone information. Where a rezone area includes multiple parcels, it is generally proposed that the total residential units be distributed to all of the included parcels in proportion to parcel area. The unit flow factor (UFF) used for the residential development would depend on the density, as follows:

- LDR — Single-family UFF (180/190 gpd/DU for conveyance/treatment)
- MDR/HDR — Multi-family UFF (130 gpd/DU)

Two of the rezone areas (Nos. 15 and 16) are part of areas identified for redevelopment by the City of Roseville. Flows for parcels in these areas will be handled under the land use "intensification" scenario and will be addressed in a subsequent memorandum to be prepared in conjunction with the SPWA Wastewater Systems Evaluation Project.

Zoning, Land Use changes and development intesifications

1) LONGMEADOW --AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, FROM INDUSTRIAL TO 145 LDR RESIDENTIAL UNITS ON 32.9 ACRES, 399 MDR RESIDENTIAL UNITS ON 45.8 ACRES, 3 ACRES OF PARK AND 7.6 ACRES OF OPENSPACE
SUBMITTED 6/12/03
APPROVED 4/7/04

2) FIDDYMENT (WALAIRE) 44 RZ 03-09
AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, AND ESTABLISH A DEVELOPMENT AGREEMENT PROVIDES FOR 142 RESIDENTIAL UNITS ON 44.53 ACRES
SUBMITTED 9/4/03
APPROVED 4/6/05

3) NWRSP PCL 77 ROSEPARK RZ 03-12
TO REZONE FROM BUSINESS PROFESSIONAL (BP/SA-NW) TO MEDIUM DENSITY RESIDENTIAL (MDR 8.24) PROVIDES FOR 86 LDR RESIDENTIAL UNITS, OPEN SPACE (OS) AND PQP
SUBMITTED 12/5/03
APPROVED 7/7/04

4) NWRSP 37 -LEGACY--AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, FROM COMMERCIAL TO 71 LDR RESIDENTIAL UNITS ON 10.4 ACRES.
SUBMITTED 11/07/03
APPROVED 10/06/04

5) NCRSP 18C AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, FROM BUSSINESS PROFESSIONAL TO 249 MDR RESIDENTIAL UNITS ON 26.43 ACRES.
SUBMITTED 7/01/03
APPROVED 2/18/04

6) CHURCH STREET STATION
CHANGE ZONING FROM M1 TO R3 PROVIDES FOR 48 LDR RESIDENTIAL UNITS ON 4 ACRES.
SUBMITTED 7/11/03
APPROVED 8/4/04

7) NERSP PARCEL 16 STONEPOINTE AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, FROM RESEARCH AND DEVELOMENT TO 575 MDR AND HDR RESIDENTIAL UNITS ON 44 ACRES.
SUBMITTED 4/2/03
APPROVED 3/16/05

8) NWRSP PCL 11 RZ 04-04
MODIFY ZONING FROM COMMUNITY COMMERCIAL TO 6.7 ACRES COMMUNITY COMMERCIAL (CC/SA/-NW) AND 6.7 ACRES ATTACHED HOUSING RESIDENTIAL (R3) –PROVIDES FOR 53 RESIDENTIAL UNITS AND 4.0 ACRES COMMUNITYY COMMERCIAL (CC/SA-NW)
SUBMITTED 9/2/04
APPROVED 7/20/05

9) NCRSP PCL 44 RZ 04-08
20.44 ACRES FROM BUSINESS PROFESSIONAL/SPECIAL AREA-NORTH CENRAL (BP/SA-NC) TO R3 (ATTACHED HOUSING-MDR12) PROVIDES FOR 244 RESIDENTIAL UNITS.
SUBMITTED 12/23/2004
APPROVED 07/20/05

10) HP AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, LIGHT INDUSTRIAL TO 794 HDR UNITS ON 43.7 ACRES, 1,203 MDR UNITS ON 119.4 ACRES, 2 LDR UNIS ON .7 ACRES, COMMUNITY COMMERCAIL ON 11 ACRES, BUSINESS PROFESIONAL ON 50 ACRES, PUBLIC QUASI PUBLIC ON 16.2 ACRES AND 45.9 ACRES OF OPEN SPACE.
SUBMITTED.9/22/05, NO ACTION TAKEN YET

11) NRSP PCL 41 AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, FROM COMMUNIT COMMERCIAL TO 125 MDR RESIDENTIAL UNITS ON 9.7 ACRES.
SUBMITTED 12/29/04
APPROVED NO ACTION TAKEN YET

12) NIRSP DIAMOND PLAZA CONDOMINIUMS AMEND THE GENERAL PLAN LAND USE DESIGNATION, REZONE, FROM GENERAL COMMECAIL TO 202 LDR RESIDENTIAL UNITS ON 13.4 ACRES.
SUBMITTED 3/28/04
NO ACTION TAKEN YET

13) DIAMOND CREEK AMEND THE GENERAL PLAN LAND USE DESIGNATION ON 19 ACRES REZONING, TO RESIDENTIAL.
NO APPLICATION RECIEVED ON THIS REQUEST

14) NCRSP PCL 40 SPA 04-06
INCREASE IN FAR FROM .4 TO .8 FAR NCRSP PARCELS 40, A,B,C ALLOWING BUILDINGS UP TO 10 STORIES ON 50 ACRES
SUBMITTED 12/23/2004
APPROVED 07/20/05

15) 416 RIVERSIDE (RIVERSIDE CORRIDOR) RZ 04-07

TO DEVELOP A SPECIFIC PLAN DOCUMENT FOR THE RIVERSIDE GATEWAY PROJECT, THE PROJECT WILL ADD 550,000 SQ FT OF MIXED USE COMMERCIAL AND RESIDENTIAL ON 21 ACRES.
SUBMITTED 11/05/04
NOT YET ACTED UPON

16) CIVIC PLAZA PROJECT RZ 04-02
TO REZONE PROPERTY FROM PLANNED DEVELOPMENT (PD 3272) & COMMERCIAL BUSINESS DISTRICT (CBD) TO COMMERCIAL BUSINESS DISTRICT (CBD) PROVDES FOR A 56,000 SQUARE FOOT BUILDING WITH COMMERCAIL GROUND FLOOR AND OFFICES ABOVE ALONG WITH A 5 LEVEL PARKING GARAGE ON 4 ACRES.
SUBMITTED 6/4/04
APPROVED 7/6/05

17) SOUTH PLACER JUSTICE CENTER JUSTICE CENTER INCLDING A COURT HOUSE AND JAIL WITH ASSOCAITED ADMINSTRATIVE BUILDNGS AND OFFICES TOTALING 679,000 SQUARE FEET ON 69 ACRES.
APPROVED 12/11/03

18) KAISER MPP 02-02 HOSPITAL EXPANSION TOTALING 1.08 MILLION SQUARE FEET ON 48 ACRES
APROVED 3/11/04

19) GALLERIA MALL EXPANSION MPP 04-02 ADDITION OF 479,000 SF TWO-LEVEL EXPANSION OF NEW RETAIL SHOPS WILL RESULT IN A TOTAL GROSS SQUARE FOOTAGE OF 1.8 MILLION SQUARE FEET ON 95 ACRES.
NOT YET ACTED UPON

Roseville Rezone Parcels

Rezone Area	APN	Parcel Area (ac.)	Current Database Existing Land Use	Current Database Future Land Use	Proposed Rezone Land Use	DUs	Comments
1	017-115-083-000	17.5	UNCONNECTED	LIGHT INDUSTRIAL	RESIDENTIAL 100 DU	100	544 DUs (LDR/MDR) on 4 parcels; apply **145 gpd/DU** weighted SF/MF UFF.
1	017-115-084-000	14.3	UNCONNECTED	LIGHT INDUSTRIAL	RESIDENTIAL 81 DU	81	
1	017-115-085-000	24.5	UNCONNECTED	LIGHT INDUSTRIAL	RESIDENTIAL 139 DU	139	
1	017-115-086-000	39.2	UNCONNECTED	LIGHT INDUSTRIAL	RESIDENTIAL 224 DU	224	
2	017-115-018-000	44.5	UNCONNECTED	LIGHT INDUSTRIAL	RESIDENTIAL 142 DU	142	142 DUs (**SF UFF**)
3	017-370-019-000	13.4	UNCONNECTED	RESIDENTIAL 91 DU	RESIDENTIAL 86 DU	86	86 DUs (**MF UFF**) plus OS and PQP; ignore PQP (probably small area)?
4	015-350-022-000	9.6	UNCONNECTED	COMMERCIAL	RESIDENTIAL 71 DU	71	71 DUs (**SF UFF**)
5	363-020-036-000	26.4	UNCONNECTED	COMMERCIAL	RESIDENTIAL 249 DU	249	249 DUs (**MF UFF**)
6	012-184-008-000	1.4	UNCONNECTED	LIGHT INDUSTRIAL	RESIDENTIAL 24 DU	24	48 DUs (**MF UFF**) split between 2 parcels
6	012-184-009-000	1.5	UNCONNECTED	LIGHT INDUSTRIAL	RESIDENTIAL 24 DU	24	
7	048-460-018-000	1.9	UNCONNECTED	COMMERCIAL	RESIDENTIAL 33 DU	33	575 DUs (**MF UFF**) on 5 parcels
7	048-460-019-000	7.3	UNCONNECTED	COMMERCIAL	RESIDENTIAL 130 DU	130	
7	048-460-020-000	7.8	UNCONNECTED	COMMERCIAL	RESIDENTIAL 139 DU	139	
7	048-460-021-000	7.6	UNCONNECTED	COMMERCIAL	RESIDENTIAL 135 DU	135	
7	048-460-022-000	7.8	UNCONNECTED	COMMERCIAL	RESIDENTIAL 138 DU	138	
8	477-080-005-000	10.7	UNCONNECTED	COMMERCIAL	Breton Village		53 DUs (SF UFF) plus 4 acres commercial; calculate flow as point source = **13,000 gpd**
9	363-010-011-000	20.4	UNCONNECTED	COMMERCIAL	RESIDENTIAL 244 DU	244	244 DUs (**MF UFF**)
10	017-230-051-000	287*	HP	HP	HP		* Northern and western portion of parcel (Areas A and B, not including existing campus). 1,997 MF Dus; 2 SF Dus; 11 acres commercial; 50 acres business professional; 16.2 ac. PQP; 45.9 acres open space. Calculated flow = **320,000 gpd** (rezone area). Add future flow from existing campus = 270,000 gpd (per Art O'Brien). Total HP point source flow = **590,000 gpd**.
11	017-162-069-000	9.7	UNCONNECTED	COMMERCIAL	RESIDENTIAL 125 DU	125	125 DUs (**MF UFF**)
12	017-410-019-000	2.3	UNCONNECTED	COMMERCIAL	RESIDENTIAL 28 DU	28	202 DUs (**MF UFF**) on 7 parcels
12	017-410-020-000	2.0	UNCONNECTED	COMMERCIAL	RESIDENTIAL 24 DU	24	
12	017-410-021-000	2.2	UNCONNECTED	COMMERCIAL	RESIDENTIAL 26 DU	26	
12	017-410-022-000	2.0	UNCONNECTED	COMMERCIAL	RESIDENTIAL 24 DU	24	
12	017-410-035-000	5.0	UNCONNECTED	COMMERCIAL	RESIDENTIAL 59 DU	59	
12	017-410-041-000	1.6	COMMERCIAL	COMMERCIAL	RESIDENTIAL 19 DU	19	
12	017-410-042-000	1.8	UNCONNECTED	COMMERCIAL	RESIDENTIAL 22 DU	22	
13	017-115-032-000	19.0	UNCONNECTED	COMMERCIAL	RESIDENTIAL 285 DU	285	50/50% MDR and HDR. Assume average 15 DU/ac, total 285 DUs (**MF UFF**)
14	363-010-009-000	50.0	UNCONNECTED	COMMERCIAL	Convention Center		Increase from 0.4 to 0.8 FAR; use 3,300 gpd/ac UFF (~4 x commercial UFF) = **165,000 gpd**
15	014-011-004-000	0.3	COMMERCIAL	COMMERCIAL	TBD		Most of Upper Riverside redevelopment area; ~0.8 FAR; **handle as intensification area**
15	014-011-008-000	0.4	COMMERCIAL	COMMERCIAL	TBD		
15	014-011-009-000	0.3	COMMERCIAL	COMMERCIAL	TBD		
15	014-013-002-000	0.1	RESIDENTIAL 2 DU	RESIDENTIAL 2 DU	TBD		

Roseville Rezone Parcels

Rezone Area	APN	Parcel Area (ac.)	Current Database Existing Land Use	Current Database Future Land Use	Proposed Rezone Land Use	DUs	Comments
15	014-013-003-000	0.1	RESIDENTIAL 3 DU	RESIDENTIAL 3 DU	TBD		
15	014-013-004-000	0.1	MIXED USE	MIXED USE	TBD		
15	014-013-005-000	0.2	MIXED USE	MIXED USE	TBD		
15	014-013-006-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-013-007-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-013-008-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-013-009-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-013-010-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-013-011-000	0.1	RESIDENTIAL 1 DU	RESIDENTIAL 1 DU	TBD		
15	014-024-003-000	0.2	UNCONNECTED	COMMERCIAL	TBD		
15	014-024-004-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-024-005-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-024-006-000	0.2	RESIDENTIAL 2 DU	RESIDENTIAL 2 DU	TBD		
15	014-024-007-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-024-008-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-024-009-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-024-010-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-033-005-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-033-007-000	0.4	COMMERCIAL	COMMERCIAL	TBD		
15	014-033-008-000	0.5	COMMERCIAL	COMMERCIAL	TBD		
15	014-033-009-000	0.4	COMMERCIAL	COMMERCIAL	TBD		
15	014-033-014-000	0.4	RESIDENTIAL 1 DU	RESIDENTIAL 1 DU	TBD		
15	014-033-028-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-033-029-000	0.2	UNCONNECTED	COMMERCIAL	TBD		
15	014-053-003-000	0.3	COMMERCIAL	COMMERCIAL	TBD		
15	014-053-004-000	0.2	RESIDENTIAL 2 DU	RESIDENTIAL 2 DU	TBD		
15	014-053-005-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-053-006-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-053-007-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-053-008-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-053-023-000	0.5	MIXED USE	MIXED USE	TBD		
15	014-091-013-000	0.1	RESIDENTIAL 1 DU	RESIDENTIAL 1 DU	TBD		
15	014-091-014-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-091-015-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-091-016-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-091-017-000	0.2	RESIDENTIAL 3 DU	RESIDENTIAL 3 DU	TBD		
15	014-091-018-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-091-019-000	0.3	UNCONNECTED	COMMERCIAL	TBD		
15	014-091-020-000	0.2	COMMERCIAL	COMMERCIAL	TBD		

Roseville Rezone Parcels

Rezone Area	APN	Parcel Area (ac.)	Current Database Existing Land Use	Current Database Future Land Use	Proposed Rezone Land Use	DUs	Comments
15	014-091-021-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-091-022-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-012-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-013-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-014-000	0.2	UNCONNECTED	COMMERCIAL	TBD		
15	014-141-015-000	0.2	UNCONNECTED	COMMERCIAL	TBD		
15	014-141-016-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-017-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-019-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-020-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-021-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-022-000	0.2	RESIDENTIAL 1 DU	RESIDENTIAL 1 DU	TBD		
15	014-141-023-000	0.3	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-026-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-141-027-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
15	014-191-016-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-191-017-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-191-018-000	0.2	UNCONNECTED	COMMERCIAL	TBD		
15	014-191-019-000	0.2	MIXED USE	MIXED USE	TBD		
15	014-191-020-000	0.2	RESIDENTIAL 2 DU	RESIDENTIAL 2 DU	TBD		
15	014-191-021-000	0.3	COMMERCIAL	COMMERCIAL	TBD		
15	014-191-022-000	0.3	COMMERCIAL	COMMERCIAL	TBD		
15	014-191-023-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-191-024-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-191-025-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-251-011-000	0.4	COMMERCIAL	COMMERCIAL	TBD		
15	014-251-012-000	0.4	LIGHT INDUSTRIAL	LIGHT INDUSTRIAL	TBD		
15	014-251-013-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-251-014-000	0.2	COMMERCIAL	COMMERCIAL	TBD		
15	014-251-015-000	0.1	UNCONNECTED	COMMERCIAL	TBD		
15	014-251-016-000	0.1	RESIDENTIAL 1 DU	RESIDENTIAL 1 DU	TBD		
15	014-251-021-000	0.4	COMMERCIAL	COMMERCIAL	TBD		
15	014-251-027-000	1.2	COMMERCIAL	COMMERCIAL	TBD		
16	013-123-001-000	0.1	COMMERCIAL	COMMERCIAL	TBD		Small part of Vernon redevelopment area; 56,000 sq ft com/off bldg w/garage; **handle as intensification area**
16	013-123-002-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
16	013-123-003-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
16	013-123-004-000	0.1	COMMERCIAL	COMMERCIAL	TBD		
17	017-122-001-000	67.0	UNCONNECTED	HEAVY INDUSTRIAL	PUBLIC/QUASI-PUBLIC		South Placer Justice Center. Use standard **PQP UFF**.
18	048-010-030-000	49.2	Kaiser Hospital	Kaiser Hospital	Kaiser Hospital		Expansion from 378K to 1.08M sq. ft. Increase flow in proportion to expansion square footage. Per City, future flow = **118,000 gpd**.

Roseville Rezone Parcels

Rezone Area	APN	Parcel Area (ac.)	Current Database Existing Land Use	Current Database Future Land Use	Proposed Rezone Land Use	DUs	Comments
19	363-010-025-000	1.8	COMMERCIAL	COMMERCIAL	Galleria Mall		Galleria Mall expansion. Total 1.8M sq. ft. Per City, future flow = 238,250 gpd. Distribute to parcels based on **2,700 gpd/acre** UFF.
19	363-010-027-000	1.6	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-028-000	4.3	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-029-000	7.6	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-030-000	6.0	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-031-000	4.1	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-032-000	9.9	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-033-000	1.3	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-034-000	7.6	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-035-000	1.3	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-036-000	12.5	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-037-000	1.6	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-038-000	2.2	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-039-000	1.4	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-040-000	2.3	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-041-000	4.2	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-042-000	0.4	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-043-000	7.8	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-044-000	2.4	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-045-000	4.3	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-061-000	1.6	COMMERCIAL	COMMERCIAL	Galleria Mall		
19	363-010-062-000	0.8	COMMERCIAL	COMMERCIAL	Galleria Mall		

Abbreviations:

DU	Dwelling unit
UFF	Unit flow factor
SF	Single family
MF	Multi-family
CC	Commercial
FAR	Floor-area ratio
TBD	To be determined